ANALYSIS OF SLOPE STABILITY FOR WASTE DUMPS IN A MINE

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF

Bachelor of Technology in Mining Engineering

by

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2011

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NATIONAL INSTITUTE OF TECHNOLOGY

ROURKELA

CERTIFICATE

This is to certify that the thesis entitled, "**Analysis of Slope Stability for Waste Dumps in a Mine**" submitted by **Mr Gorakinkar Das, Roll No. 107MN034** in partial fulfilment of the requirement for the award of Bachelor of Technology Degree in Mining Engineering at the National Institute of Technology, Rourkela (Deemed University) is an authentic work carried out by him under my supervision and guidance.

To the best of my knowledge, the matter embodied in the thesis has not been submitted to any University/Institute for the award of any Degree or Diploma.

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Date: **Gorakinkar Das**

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ABSTRACT

In the civilized world mining activities are synonymous with the standard of life as well as the state of any nation. It results in both economic and uneconomic materials being generated. The Uneconomic materials (Wastes) are stacked at different places known as waste dumps. The stability of these dumps has been a major concern over the years. The problem becomes increasingly difficult with the reduced availability of land areas for dumping. In this project, the slope stability analysis for the waste dump of a local Iron Mine has been carried out. Samples are collected and tasted in the laboratory to find out different geo-technical parameters. The Factors of Safety of the various sections are calculated using Limit Equilibrium Method. Probabilistic analysis (Monte Carlo Simulation) has also been carried out to evaluate the stability of the existing design data. At the end bench design as well as corresponding safety factors has been developed based on the analysis.

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CHAPTER - 1

INTRODUCTION

1.1 Background of the problem

In the modern world mining has become an essential act for the production of economic minerals. In the production process huge amount of wastes are generated. These waste materials are stacked in a convenient place for its further use or disposal, or are stored permanently. They are stored in the form of a slope or embankment. In either of the purposes the stability of the slope has been a major concern.

The region affected by the slope failure is always not necessarily the slope's immediate area. The stability of sloped land areas, landslide, is a main concern where movements of existing or planned slopes would have an effect on the safety of people and property or the usability and value of the area.

Constantly appearing disasters have a great weight on the issue of slope stability. The disasters and devastation include the natural events (torrential rains), uncased excavations, road embankments and landfills. The quoted phenomena take place due to either an incorrect approach to the assessment of their stability, or mistakes made at the stage of geotechnical investigations, erroneous assumptions made in the phase of carrying out calculation, or an improper location of machines on the slope surcharge.

One of the causes of the incorrect assessment of slope stability may be inaccurate determination of the geological structure of the slope in question. As the mine expands over a period of time, these waste dumps and the issues regarding their stability become important. To deal with these slope stability issues various approaches have been adopted and developed over the years. The approaches now have been more of computational rather than the manual. Various software are available to analyze the slopes that are liable to failure by the calculating the factor of safety.

Of particular relevance to slope stability analysis are the finite elements and limit equilibrium methods. However, when using limiting equilibrium methods to analyze slopes, several numerical inconsistencies and computational difficulties may occur in locating the critical slip surface (depending on the geology) and hence establishing a factor of safety. Despite these inherent limitations, due to its simplicity limiting equilibrium continues to be the most commonly used approach. [*N. A. Hammouri et al.* 2008]

Fig 1.1: The crest of a typical dump failure. *(Source: E. Steiakakis et al.,* 2009*)*

1.2 Aim of the Study

The objective of this project is to investigate the stability of the slopes by determining the factor of safety and to propose different safe slopes. This has been achieved by the following specific objectives.

1.2.1 Specific Objectives:

The primary objective of this project is to evaluate the stability of the slopes constructed due to dumping of the waste materials (especially the sub-grade materials). It has the following specific objectives:

- Critical review of the available literature to understand the issues involved.
- Collection of samples from field or an operating mine.
- Analysis of the samples to find out parametric variations (as the cohesion, angle of friction, density, moisture content, grain sizes, etc.) affecting the stability.
- Prediction of safety factor based on the geotechnical data.
- Determination of Reliability Index using Monte Carlo Simulation.
- Valuation of the safety factors and suggestion of alternate geometry.

1.3 Methodology:

The methodology for this project is grouped into several stages as shown below.

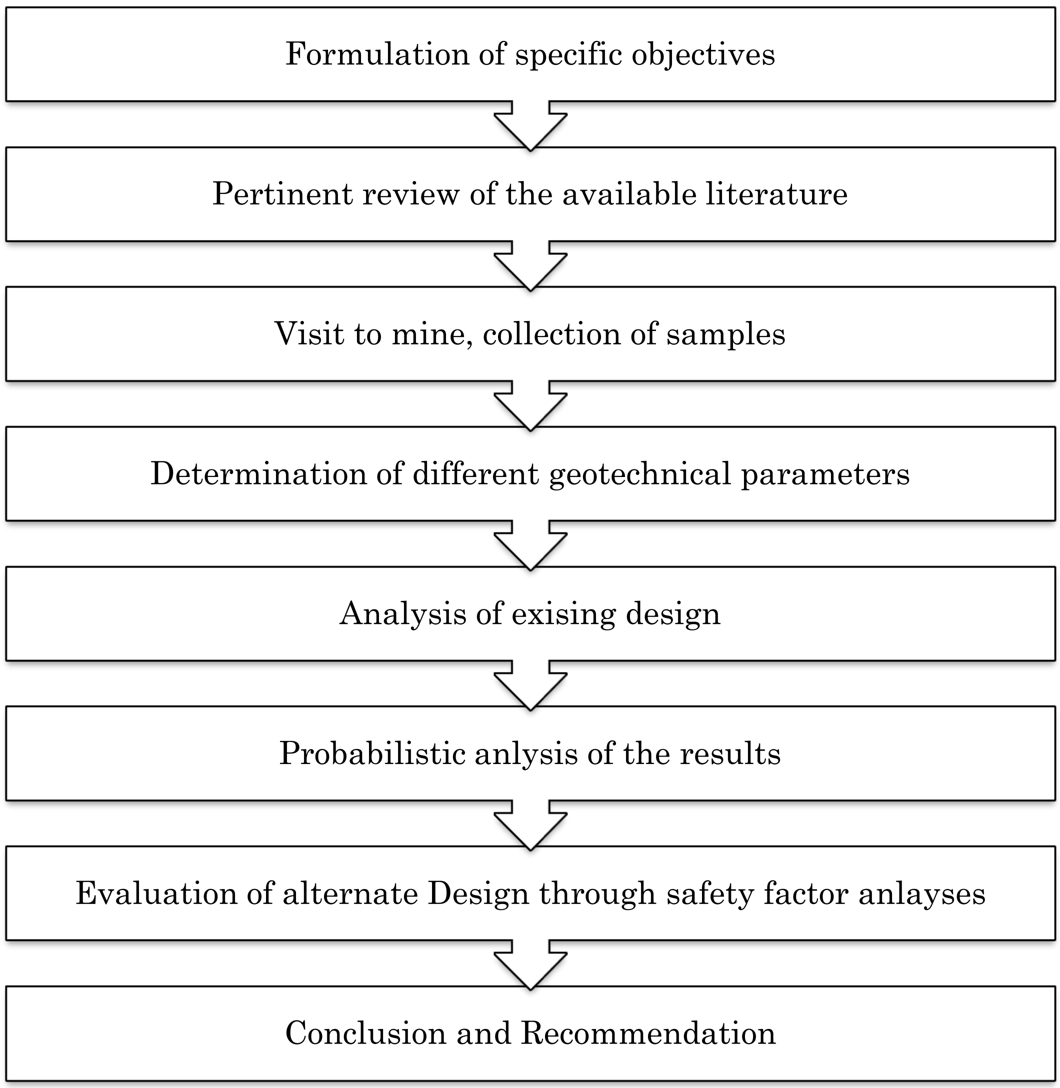


Fig 1.2: Methodology of the research.

Probabilistic approach using Monte Carlo Simulation is carried out by the simulation of the results according to number of iterations. It is done by generating 100, 500, 1000, 200, 3000, 4000, 5000, 6000, 7000, 8000, 9000 and 10000 numbers of samples.

CHAPTER - 2

LITERATURE REVIEW

2.1 Stability Analysis – General Concepts *(McCarthy & David,* 2007*)*

The **slope stability analyses** are generally performed to appraise the safe and economic design of human-made or natural slopes (e.g. embankments, open-pit mining, excavations, landfills etc.) and the equilibrium conditions. The term 'slope stability' may be defined as the ratio of the resistance of inclined surface to failure by sliding or collapsing. The main objectives of slope stability analysis are ascertaining endangered areas, investigating potential failure mechanisms, finding of the slope sensitivity to different triggering mechanisms, designing of optimal slopes with regard to safety, reliability and economics, designing possible remedial measures, e.g. barriers and stabilization.

Where the stability of a sloped earth mass is to be studied for the possibility of failure by sliding along a circular slip surface, the principles of engineering statics can be applied to determine if a stable or unstable condition exists. When the total sliding mass is assumed to be cylindrical or spoon-shaped, a unit width extending along the face of the slope is taken for analysis, and the slip surface of the slope cross section is the segment of a circle. Forces that would affect the equilibrium of the assumed failure mass are determined, and rotational moments of these forces with respect to a point representing the center of the slip circle arc (the point is actually an axis in space parallel to the face of the slope) are computed. With this procedure, the weight of soil in the sliding mass being considered as well as external loading on the face and top of the slope contribute to the moments acting to cause movement. Resistance to sliding is provided by the shear strength of the soil on the assumed slip surface.

A computational method used to indicate if failure (sliding) occurs is to compare moments that would resist movement to those that tend to cause movement. The maximum shear strength possessed by the soil is used in the calculation of the resisting moment. Failure is indicated when moments causing motion exceed those resisting motion. The factor of safety against sliding or movement is expresses as:

$$F = \frac{\text{Moments resisting sliding}}{\text{Moments causing sliding}}$$

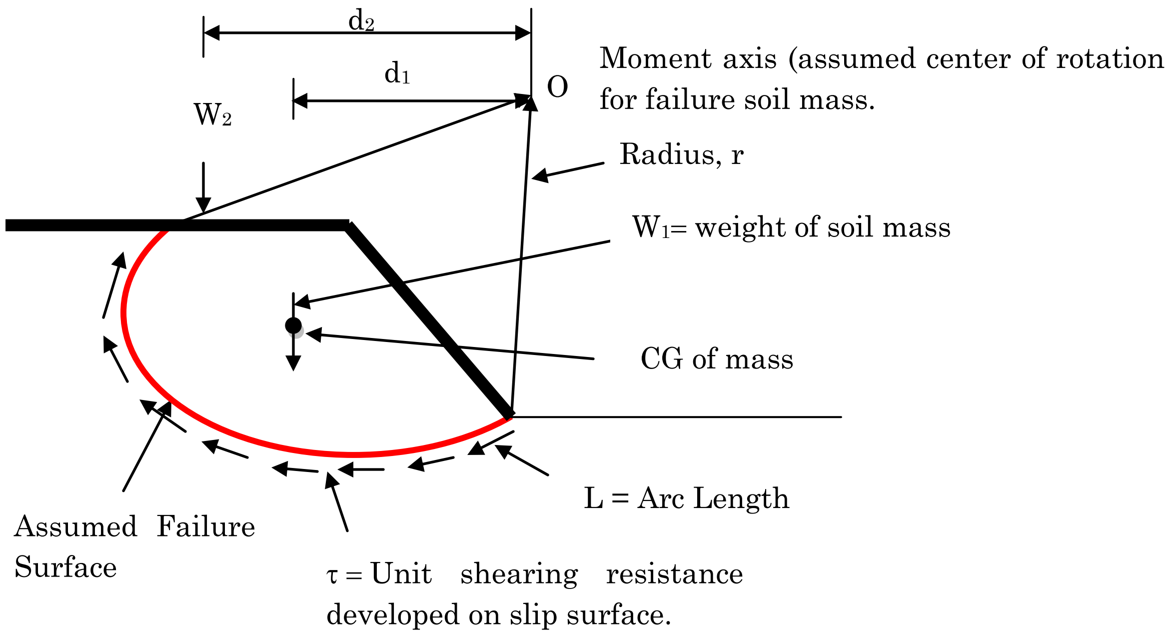


Fig 2.1: Description of the forces acting on an assumed slope failure mass

Here, W_2 = External loading on failure area.

d_1 = Distance between Moment axis and CG of mass.

d_2 = Distance between Moment axis and failure surface.

Moment causing sliding = $(W_1 \times d_1) + (W_2 \times d_2)$

Moment resisting sliding = $\tau \times L \times r$

Hence, Factor of Safety (F) = $\frac{\text{Moment resisting sliding}}{\text{Moment causing sliding}}$

$$F = \frac{\tau \times L \times r}{(W_1 \times d_1) + (W_2 \times d_2)}$$

A factor of safety of unity implies that the assumed failure mass is on the verge of sliding. A variation to this method for studying slope stability involves determining the shear strength required to have sliding moments and resisting moments balance (equilibrium). The shearing resistance required along the slip surface is compared to the shear strength that can be developed by the soil. If the soil shearing strength that can be developed by the soil is greater than the shearing resistance required for equilibrium, failure does with this method, the factor of safety against sliding is:

$$F = \frac{\text{Shear strength possessed by the soil}}{\text{Soil shear strength required for equillibrium}}$$

The factor of safety indicated by this method is a value based on the soil's shear strength. This method is used in most of the mathematical slope stability theories.

Table 2.1: Minimum accepted values for factor of safety for slopes (*Priest & Brown*, 1983)

Category of Slope	Consequences of Failure	Examples	Minimum Factor of Safety
1	Not serious	Individual benches, small* temporary slopes not adjacent to haulage roads	1.3
2	Moderately serious	Any slope of a permanent or semi-permanent nature	1.6
3	Very Serious	Medium sized and high slopes carrying major haulage roads or underlying permanent installations	2.0

*Small height < 50m; medium height 50 to 150m; height > 150m

2.2 Factors affecting slope stability *(McCurthy & David,* 2007*)***:**

Many factors affect the stability of any slope. Those are

1. Gravitational Force.
2. Material properties of the slope.
3. Geology and hydrogeology of the dumping area.
4. Inclination of the dumping area.
5. Erosion of the surface slopes due to flowing water.
6. The sudden lowering of water adjacent to a slope.
7. Forces due to earthquakes.

The effect of all the movements is caused by the soil to move from high points to low points. The component of the gravitational force that acts in the direction of probable motion is considered to be the most important one.

Though the effects of flowing or seeping water are generally recognized as very important aspects in stability problems, but these problems have not been properly identified. The main factor being the seepage occurring within a soil mass causes seepage forces, which have much greater effect than is commonly realized.

As far as mass movement is concerned, erosion on the surface of the slope may remove a certain weight of soil, and may thus lead to an increased stability. On the other hand, undercutting of erosion at the toe may increase the height of the slope or decrease the length of the incipient failure surface, thus decreasing the stability.

Lowering of the ground-water surface or of a free water surface adjacent to the slope leads to diminish the buoyancy of the soil which is in the effect an increase in the weight. The increase in weight results in increase in the shearing stresses, whether or not the value of permeability is lower. Practically no changes in volume will take place except at a certain slope rate, and in spite of the increase of load, increase in strength may be inappreciable.

A decrease in the inter-granular pressure and increase in the neutral pressure accompany shear force at a constant volume. A different condition may exist in which the soil mass converts into a state of liquefaction and flows like liquid. This type of condition is likely to be developed if the mass of the soil is subjected to vibration, possibly due to earthquake.

2.3 Sliding Block Analysis (*McCurthy & David, 2007*) (Fig 2.2 and 2.3)

Slopes consisting of the stratified materials and embankment structures on the constructed on the stratified soil foundations can experience failure due to the sliding along one or more of weaker layers. This type of failure often occur when changed conditions in an area cause the susceptible layers to become exposed to, or saturated by, water. Exposure to moisture can cause physical breakage and weakening of some earth materials, such as fine grained sedimentary deposits, and saturation may cause reduction in stratum's shear strength because of the increase in pore water pressures.

Where the potential for the occurrence of a block slide is under the study with no pore pressure effect on the block, the factor of safety with regard to the shear strength of the soil on the assumed sliding plane is given by,

$$F = \frac{cL + W(\cos\alpha + E\sin\alpha)}{(w\sin\alpha + E\cos\alpha)}$$

Where E can be approximated as 0.25 γ_{soil} Z^2 for cohesion less soil and 0.5 γ_{soil} Z^2 for cohesive soil if the formation of a tension crack along the top of the slope permits the development of water pressure in the crack and the slippage zone, can be described as:

$$F = \frac{(cL + (W\cos\alpha - F_u + F_w\sin\alpha)\tan\Phi)}{W\sin\alpha + F_w\cos\alpha}$$

Where F_w is the force due water pressures in the tension crack, equal to $0.5\gamma_{water}\ Z^2_{water}$ and $F_u = 0.5\gamma_{water}\ W_{water}\ L$.

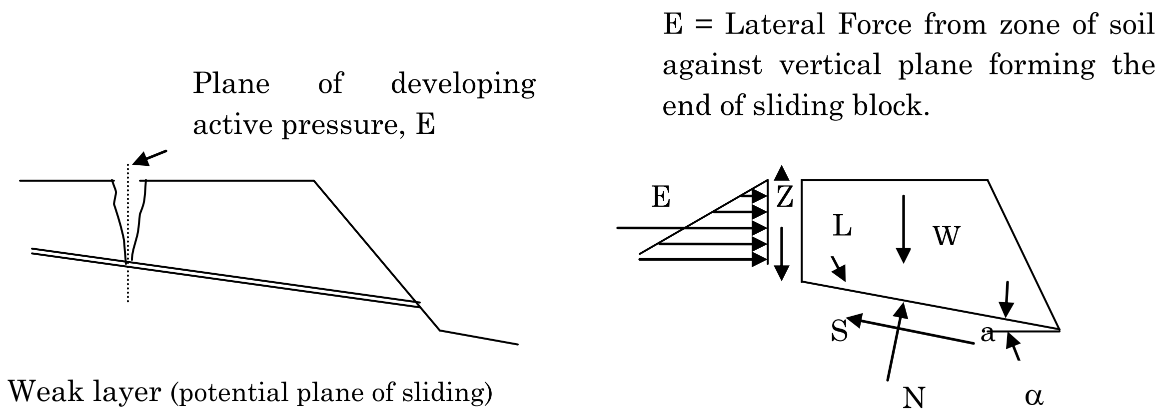


Fig 2.2: Contribution to failure along weak plane by active pressure zone at top sliding block

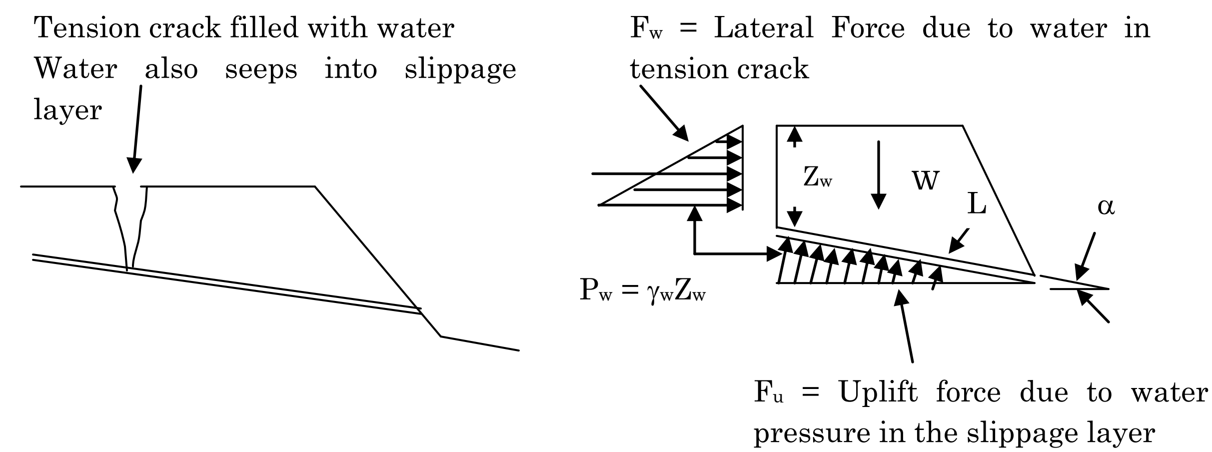


Fig 2.3: Failure along a weak plane where water pressure develops in the tension crack and slippage layer

Sections of several slopes have known to fail by translation along a weak foundation zone or layer, the force responsible for movement responsible for movement resulting from lateral soil pressure developed within the embankment itself. In the case of the earth dams, the zone of the slippage may develop only after the dam has impounded water for a period, with seepage through the eventual slippage zone being responsible for weakening to the extent that a failure can occur.

The upstream as well as the downstream zones might be studied for stability. Though the effect of water on the upstream embankment increases the weight ‘W’, the lateral pressure of the impounded water for a period opposes block translation. The uplift force is considerably greater for upstream zones. Like other categories of sliding failures, it determines the size and location of the section most susceptible to movement. It is typically a trial and error procedure, because the most critical zone is not always obvious.

2.4 Phreatic Surface

The term phreatic is used in earth sciences in reference to matters relating to ground water below the water table. The term 'phreatic surface' points the location where the pore water pressure is under the condition of atmosphere (i.e. the pressure head is zero). Normally, this surface normally coincides with the water table.

2.5 Effect of Tension Cracks

Developing Tension cracks along the face or crest of a slope (a condition most often anticipated where cohesive soils exist) can influence stability. In an analysis, Soil possessing zero shearing resistance is assigned to the section of slippage plane affected by tension cracks. If the tension crack(s) could fill with water, a hydrostatic pressure distribution is assumed to exist in the crack, and this pressure contributes to those forces and moments acting to cause slope movement. Frequently, however, the computed factor of safety is less influenced by the tension cracks, but it might be affected if tension cracks provide the opportunity for water to reach otherwise buried earth layers whose strength may be weakened by such exposure, an effect requiring consideration in the slope analysis.

2.6 Limit equilibrium analysis

Limit equilibrium method first defines a proposed slip surface, then the slip surface is analyzed to obtain the factor of safety, which is defined as the ratio between forces (moments or stresses) resisting instability of the mass and those that causing instability (disturbing forces).

Two-dimensional sections are generally analyzed assuming plain strain conditions. The assumption for these methods is that the linear (*Mohr-Coulomb*) or non-linear relationships between shear strength and the normal stress on the failure surface governs the shear strengths of the materials along the potential failure surface.

Functional slope design calculates the critical slip surface where the factor of safety is found to be of lowest value. Computer programs can help locate failure surface using search optimization techniques. The program analyzes the stability of different layered slopes, embankments, and sheeting structures. Fast optimization of different slip surfaces (circular & non-circular surfaces) provides the lowest factor of safety. External forces (Earthquake effects, external loading, groundwater conditions, and stabilization forces) can also be included. The software uses solution in accordance with various methods of slice.

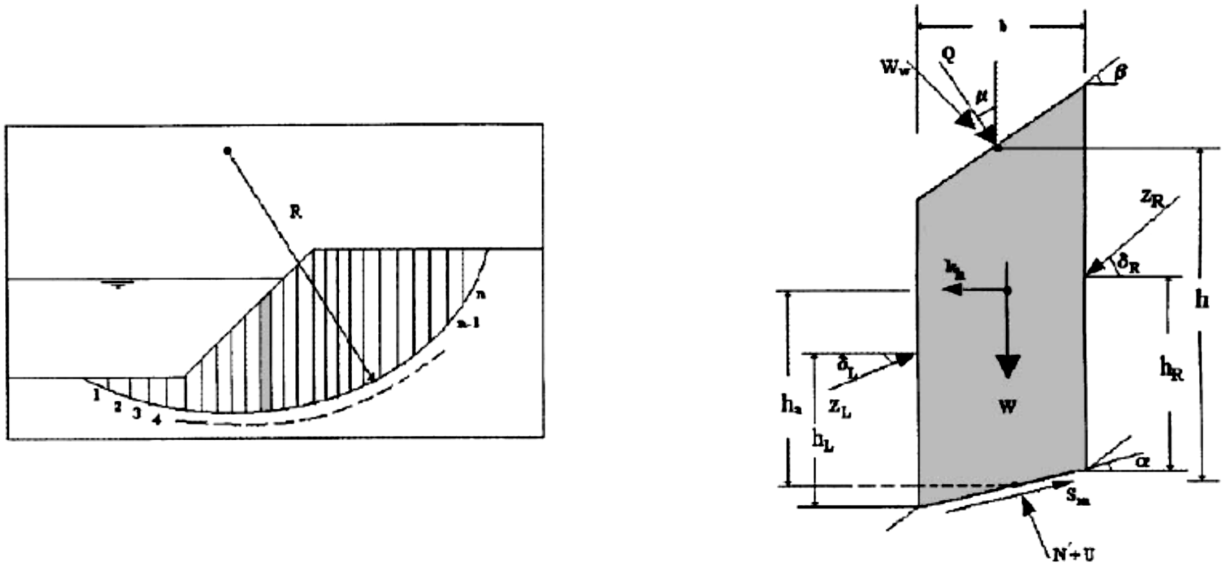
2.7 Methods of Slices (*A.I. Husein Malkawi et al,* 2000*)*

The problems associated with slope stability are statically indeterminate; hence some simplified assumptions are made in order to determine a unique factor of safety. The differences in assumptions lead to various methods of slices. The most popular methods are on the procedures proposed by Fellenius, Bishop, Janbu and Spencer. These methods either satisfy only overall moment or force equilibrium or both. The latter is applicable to failure surface of any shape. Methods like the Ordinary and simplified Bishop Methods are applicable to a circular slip surface, Janbu's method satisfies force equilibrium and is applicable to both circular and non-circular shape. Spencer's method is applicable to both moment and force equilibrium and it is applicable to failure surfaces of any shape. It is considered as one of the accurate and rigorous methods for solving stability problems.

Table 2.2: Summary of static equilibrium conditions in different limit equilibrium methods of slices (A.*I. Husein Malkawi et al,* 2000)

Method	Force equilibrium		Moment equilibrium
	1st direction	2nd direction	
Ordinary Fellenius	Yes	No	Yes
Bishop's simplified	Yes	No	Yes
Janbu's simplified	Yes	Yes	No
Spencer	Yes	Yes	Yes

In order to formulate the algorithm to solve for the factor of safety based on the above-mentioned methods, one should consider the forces acting on a typical slice as shown in Fig.



F = Factor of safety

$S_m = \frac{c`b + N` \tan \Phi}{F}$

W = Weight of slice

N` = Effective normal force

μ= Angle of inclination of external load.

Z_R = Right inter-slice force.

δ_R = Right inter-slice force inclination angle.

H_R = Height of force Z_R.

β =Inclination of slice top.

h = Average height of the slice.

S_m = Mobilized shear strength

U = Pore water pressure

W_w = Surface water force

K_h = Horizontal seismic coefficient

Z_L = Left inter-slice force.

δ_L = Left inter-slice force inclination angle.

H_L = Height of force Z_L.

α = Inclination of slice base.

b = Width of the slice

h_a = Height to the centre of the slice.

Fig 2.4: Depiction of forces acting on a typical slice (*A.I. Husein Malkawi et al,* 2000).

2.7.1 Ordinary Method of Slices

The ordinary method of slices is considered to be the simplest method of slices. The factor of safety is directly obtained in this method. The basic assumption for this method is that the inter-slice forces are parallel to the base of each slice, thus they can be neglected.

The factor of safety is given by the following equation.

$$F = \frac{\sum_{i=1}^{n}[c`b \sec\alpha + \ [W\cos\alpha + Q\ \cos(\mu - \alpha) + W_w\cos(\beta - \alpha) + K_h W \sin n\alpha - U.b] \tan \Phi`]}{\sum_{i=1}^{n}(W + W_w\cos\beta + Q\cos\mu)\sin\mu - \sum_{i=1}^{n}(W_w\sin\beta + Q\sin\mu)\left(\cos\alpha - \frac{h}{R}\right) + \sum_{i=1}^{n} K_h W\left(\cos\alpha - \frac{h_a}{R}\right)}$$

2.7.2 Bishop's simplified method

This method neglects the inter-slice forces, and hence only normal forces are used to determine the inter-slice forces. That's why Bishop's method determines the factor of safety by trial and error method.

With the Bishop method, the factor of safety appears on both sides of the equation to evaluate the stability of a trial failure mass. The procedure for solution involves *assuming* value for the factor of safety term on the right side of the equation. When the proper factor of safety has been used for the trial, the value for the left side of the equation will equal the value assumed for the right side**.** Practically, precise agreement is not required to obtain a factor of safety value considered valid for the assumed slip surface. The result is for a particular trial failure mass, however and, as indicated previously, a series of trials is usually required to identify the slope section and slippage plane most susceptible to actual failure or having the lowest factor of safety.

The Factor safety appears both sides of the equation. The Factor of safety is given as follows:

$$F = \frac{\sum_{i=1}^{n}\left[c`b\sec\alpha + \left[\frac{1}{\cos\alpha + \frac{\sin\alpha\tan\Phi`}{F}}\left[W - \frac{c`b\tan\alpha}{F} - U.b + W_w\cos\beta + Q\cos\mu\right]\right]\tan\Phi`\right]}{\sum_{i=1}^{n}(W + W_w\cos\beta + Q\cos\mu)\sin\mu - \sum_{i=1}^{n}(W_w\sin\beta + Q\sin\mu)\left(\cos\alpha - \frac{h}{R}\right)}$$

$$+\sum_{i=1}^{n} K_h W\left(\cos\alpha - \frac{h_a}{R}\right)$$

A limitation to the Bishop method is that is provides unrealistically high factors of safety when the negative angle alpha (-α) for the lower slope area slices approaches a value of about 30°. This condition can develop where an assumed center of rotation is above the vicinity of the slope's crest or where deep failure circles are being investigated.

2.7.3 Janbu's Method

In areas where there is variation in topography (the slope is not uniform or well defined) or where the subsurface is stratified or otherwise non-isotropic, the soil zone most susceptible to a sliding failure may not be properly represented by a circular arc.

Similar to Bishop's method of analysis, Janbu's method determines the factor of safety through an iterative process. The process involves the variation of normal stress on failure surface. The normal forces are generally derived from the summation of vertical forces and the inter-slice forces are ignored.

The Factor of safety can be given by:

$$F = \frac{\sum_{i=1}^{n}\left[c`b\sec\alpha + \left[\frac{1}{\cos\alpha+\frac{\sin\alpha\tan\Phi`}{F}}\left[W - \frac{c`b\tan\alpha}{F} - U.b + W_w\cos\beta + Q\cos\mu\right]\right]\tan\Phi`\right]}{\sum_{i=1}^{n}(U_b\sin\alpha + WK_h + W_w\sin\beta - Q\sin\mu) + \sum_{i=1}^{n}\left[\frac{1}{\cos\alpha+\frac{\sin\alpha\tan\Phi`}{F}}\left[W - \frac{c`b\sin\alpha}{F} - U.b\cos\alpha + W_w\cos\beta + Q\cos\mu\right]\sin\mu\right]}$$

2.7.4 Spencer's Method

The Spencer's method is considered to be the best method for finding the factor of safety. Both force and moment equilibrium is satisfied. The factor of safety is determined through an iterative procedure, slice by slice, by varying 'F' and 'δ' until force and moment equilibrium are satisfied.

The force equilibrium equation is given by:

$$Z_R = Z_L + \frac{FW\sin\alpha - c`b\sec\alpha - W\cos\alpha\tan\Phi`}{\sin(\delta-\alpha)\tan\Phi` - F\cos(\delta-\alpha)} + \frac{U.b\sec\alpha\tan\Phi` + WK_h(F - \tan\Phi`\tan\alpha)\cos\alpha}{\sin(\delta-\alpha)\tan\Phi` - F\cos(\delta-\alpha)} + \frac{Q[F\sin(\alpha-\mu) - \cos(\alpha-\mu)\tan\Phi`]}{\sin(\delta-\mu)\tan\Phi` - F\cos(\delta-\alpha)} + \frac{W_w[F\sin(\alpha-\mu) - \cos(\alpha-\mu)\tan\Phi`]}{\sin(\delta-\mu)\tan\Phi` - F\cos(\delta-\alpha)}$$

The Moment equilibrium equation is as follows:

$$h_R = \frac{Z_L}{Z_R}h_L - \frac{Z_L}{Z_R}\frac{b}{2}\tan\alpha + \frac{Z_L}{Z_R}\frac{b}{2}\tan\delta + \frac{hW_w\sin\beta}{Z_R\cos\delta} + \frac{hQ\sin\mu}{Z_R\cos\delta} - \frac{h_a k_h W}{Z_R\cos\delta}$$

The iteration is terminated when the calculated values of Z_R and h_R differ within an acceptable tolerance from the known values of Z_R and h_R at the boundary.

In general, slope stability analysis is carried out in two steps: [*Li et al*, 2005]

Step 1: Calculation for the factor of safety for a specified slip surface.

Step 2: Finding a critical failure surface that is associated with the minimum safety factor.

2.8 Probabilistic Analysis

Numbers of papers have been published on probability analysis over the decade. Though, the idea of applying probability distribution has been fascinating, still most of the geotechnical engineers regard the subject of probability theory with doubt and suspicion.

2.8.1 Monte Carlo Simulation (*US Army Corps of Engineers*, 2006)

Monte Carlo simulation is a reliability analysis method that should be used only when the system to be analyzed becomes too complex for the use of simpler methods of reliability analysis, such as, the reliability index method. In Monte Carlo simulations, each random variable is represented by a probability density function (PDF) and repeated conventional analyses are made (iterations) by changing the values of the random variables (RV) using a random number generator. To obtain an accurate Monte Carlo simulation solution, many thousands of these conventional analyses must be performed. The simpler methods of reliability analysis should be used whenever possible as the Monte Carlo simulation is much more costly and time consuming.

Thus, in Monte Carlo simulation studies three steps are usually required, namely:

1. Determining the independent variable (input).
2. Transforming the input as independent variable (output).
3. Analyzing the output.

2.8.1.1 Probability Density Functions

When performing Monte Carlo simulations each random variable must be represented by a probability density function. In Geotechnical Engineering there are only four commonly used probability density functions: uniform distribution, triangular distribution, normal distribution, and lognormal distribution. Other probability density functions would only be used if there were test data that matched those functions. Table 2.3 lists variables that are used in slope stability analysis and the probability density functions that typically best represent those variables.

Table 2.3: PDF associated with various parameters (*Lacasse and Nadim, 1996*).

Variables	Probability Density Function
Variables That Do Not Take Negative Values	Log Normal
Unit Weights	Normal
Cohesion	Normal
Friction Angle	Normal

The different probability distribution functions used in slope stability analysis are discussed below.

Uniform Distribution: The uniform distribution (Fig 2.5) is the simplest of all distributions. All that is needed is the high and the low value. The uniform distribution gives the probability that observation will occur within a particular interval when the probability of occurrence within that interval is directly proportional to the interval length. If there is no available information, the Principle of Insufficient Reason says "the uniform distribution should be used". The uniform distribution is the distribution used to generate random numbers. It is used to highlight the fact that little is known about the parameter. It is used to model circular variables (like the direction of the wind coming from 0 to 360 degrees).

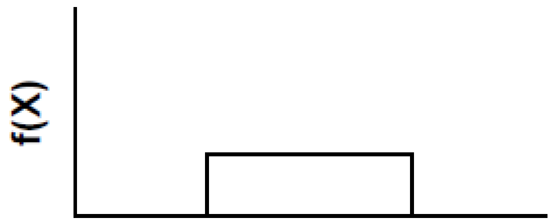


Fig 2.5: Uniform Distribution

Triangular Distribution: A triangular distribution (Fig 2.6) is used when the smallest value (most pessimistic outcome), the largest value (most optimistic outcome), and the most likely value are known. The triangular distribution is mostly used distribution for modeling expert opinion.

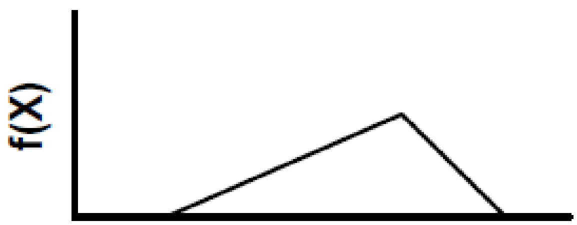


Fig 2.6: Triangular distribution

Normal Distribution: The normal distribution (Fig 2.7) is the basic distribution of statistics.

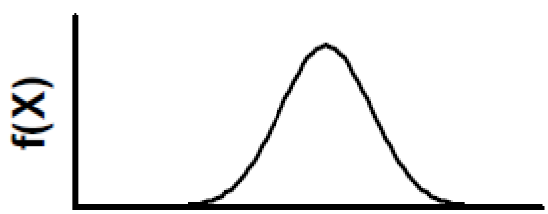


Fig 2.7: Normal distribution

The Central Limits Theorem states "the sum of many variables tends to be normally distributed". Consequently the normal distribution is an appropriate model for many but not all physical phenomena. Most things in nature tend to be normally distributed. This distribution represents physical measurements of living organisms, intelligence test scores, product dimensions, average temperatures, etc. The famous bell shape curve that students are graded on in school is the normal distribution. This distribution is easy to use because of the many standardized tables available. This is a reasonable distribution for many things. If the normal distribution is used it is hard for someone to say that it is not the correct distribution.

Lognormal Distribution: The lognormal distribution (Fig2.8) is the logarithm of the normal distribution. As such it best represents processes which are the multiplication of many variables.

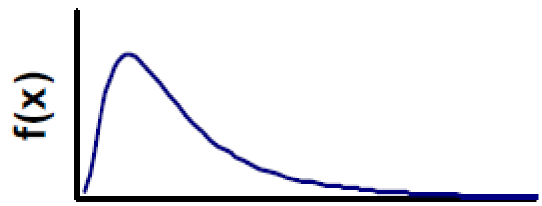


Fig 2.8: Lognormal Distribution

This distribution issued when the value of the variable cannot be less than zero. The extreme values of a normal distribution go below zero.

2.8.1.2 Random variables

Parameters such as the angle of friction, the cohesion value do not have a single fixed value but may assume any number of values. There is no way of predicting exactly what will be the value of one of these parameters at any given location. Hence, these parameters are described as random variables.

The most common graphical representation of a probability distribution is considered to be a histogram in which the fraction of all observations falling within a specified interval is plotted as a bar above that interval.

2.8.1.3 Data Analysis

To make a distribution adequate, the analysis of the various data should be done.

The sample mean or expected value or first moment is the indication of the center of gravity of a probability distribution. A typical application could be the analysis of a set of results x_1, x_2,........, x_n from a test are carried out in the laboratory. Assuming, there are '*n*' individual test values x_i, the mean x_m is given by:

$$x_m = \frac{1}{n}\sum_{i=1}^{n}(x_i)$$

The sample variance s^2 or the second moment about the mean of a distribution is defined as the mean of the square of the difference between the value of x_i and the mean value x_m.

Hence: $s^2 = \frac{1}{n-1}\sum_{i=1}^{n}(\mathrm{x} - \mathrm{x_m})^2$

2.8.2 Methodology

Monte Carlo simulations, uses the same analysis method as conventional analysis does. It runs each analysis multiple times. The random variables are represented as probability density functions. The deterministic variables are represented as constants. Monte Carlo simulation generally uses a random number generator to select the value of each random variable using the probability density function specified for that random variable. For each iteration (analysis), a factor of safety is calculated. Each factor of safety resulting from iteration is counted to develop a probability density function for the factor of safety and to determine the number of unsatisfactory performance events. The iterations are repeated thousands of times until the process converges. From the probability density function for the factor of safety or from the number of factors of safety counted that are less than one divided by the number of iterations the percent of the factor of safeties less than one is calculated to determine the probability of unsatisfactory performance which is considered as the probability of failure of a particular number of simulations. To accomplish the many thousands of iterations a computer program is needed.

Lacasse & Nadim *et al* (1996) proposed that usually normal distribution is used for various soil properties.

A.I. Husein Malkawi *et al* (2000) proposed that the reliability index (β) and the probability of failure (P_f) can be calculated using the safety factor probability distribution. This approach can be applied to any method of slices, that uses limit equilibrium in the analysis of slopes.

The uncertainty in slope stability is quantified by evaluating the reliability index, which is defined as:

$$ß = \frac{E(F) - 1}{\sigma(F)}$$

Where, ß is the reliability index, E(F) the expected value of the safety factor, and σ(F) is the standard deviation.

The Monte Carlo Simulation for this project is done by using ***GALENA*** slope stability system.

2.8.3 Flow chart for Monte Carlo simulation slope stability analysis:

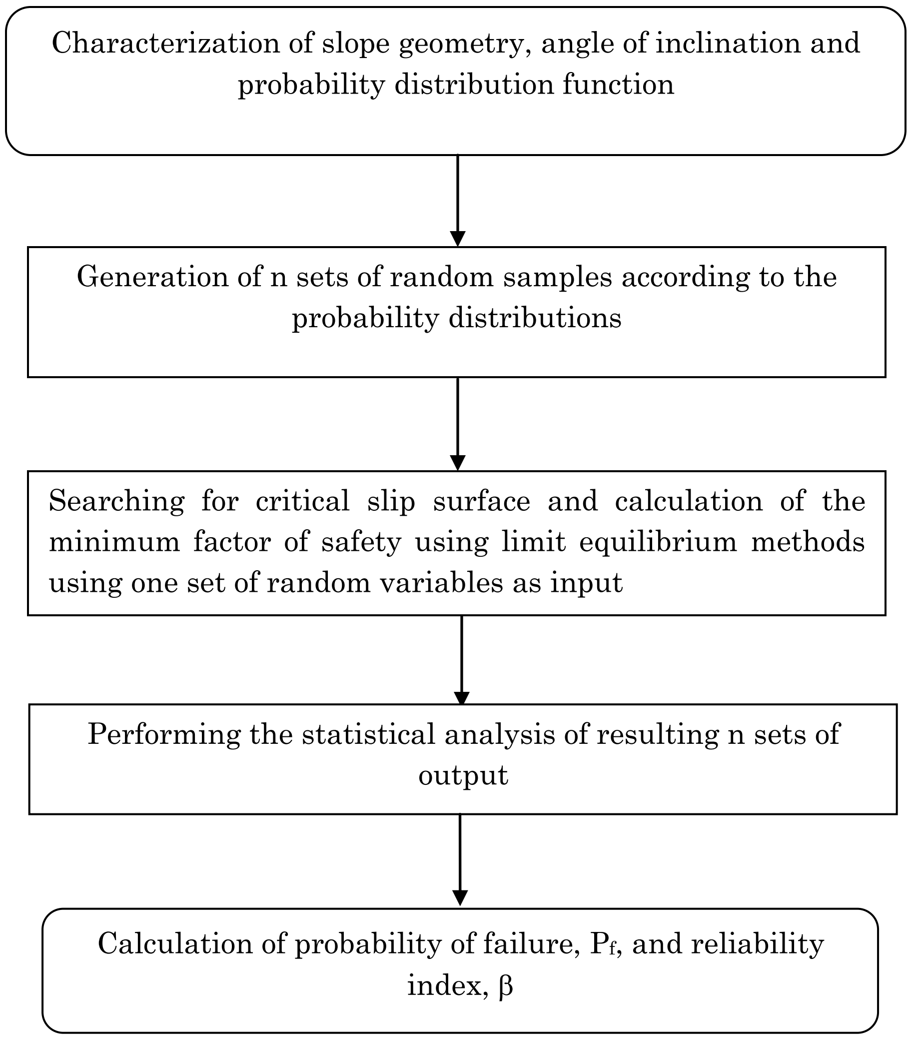


Table 2.4: Relation between reliability index, β, and probability of failure, P_f, (*U.S. Army corps of Engineers 1997*)

Reliability Index, β	Probability of failure, P_f	Expected performance level
1.0	0.16	Hazardous
1.5	0.07	Unsatisfactory
2.0	0.023	Poor
2.5	0.006	Below Average
3.0	0.001	Above Average
4.0	0.00003	Good
5.0	0.0000003	High

Wang et al (2011) reported that for a P_f level of 0.001, which corresponds to an expected performance level "above average". The sample size of direct Monte Carlo Simulation should be greater than 10,000. As the deterministic slope stability analysis explicitly searches a wide range of potential slip surfaces, it takes a considerable amount of time.

The randomness and uncertainty in the soil property are the most important factors that may affect the reliability of the safety factor.

2.9 Slope Stability Analysis System – GALENA

GALENA is designed to be a simple, user-friendly yet very powerful, slope stability software system. It was originally developed to satisfy the requirements of BHP (now known as BHP Billiton) geotechnical engineers who realized there were many problems with other slope stability analysis software systems available. Geotechnical engineering very rarely gives one unique answer and extensive parametric studies are often required before realistic results are obtained. *GALENA* enables such parametric studies to be undertaken quickly and easily.

The *GALENA* system considers slope stability problems as they are largely encountered in the field. That is, the overall geology generally remains the same; it is the slope surface that requires change in many situations. In *GALENA,* the overall geology is defined for the model, including the material properties. The defined slope surface then cuts through this model, as a slope would be excavated in the real world. Material above the slope surface is ignored since this has been removed or mined out. In this way, *GALENA* enables a large number of analyses to be undertaken without the need to redefine the model each time.

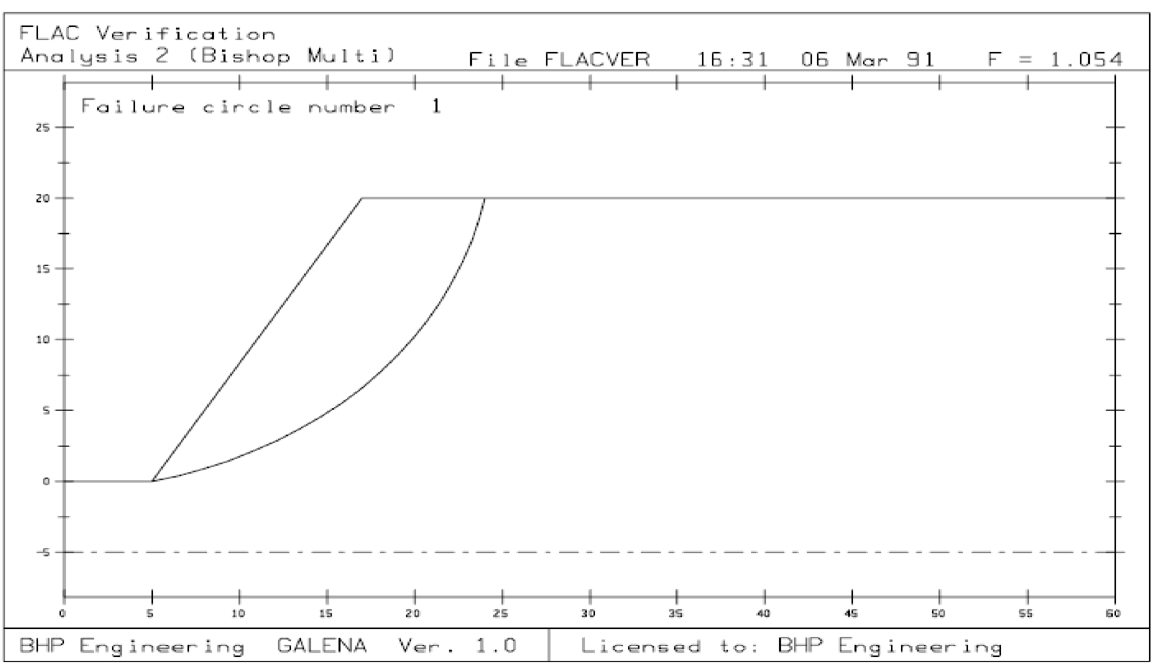


Fig 2.9: A figure depicting an analysis in earlier version of GALENA

GALENA incorporates the Bishop Simplified, the Spencer-Wright and the Sarma methods of analysis to determine the stability of slopes and excavations. The Bishop method is used to determine the stability of circular failure surfaces, the Spencer-Wright method is applicable for circular and non-circular failure surfaces, and the Sarma method is used for problems where non-vertical slices are required, or is used for more complex stability problems.

It is possible to analyze multi-layered slopes with tension cracks, earthquake forces, externally distributed loads and forces, and water pressures from within or above the slope (e.g. dams and river banks) including phreatic surfaces and piezometric pressures.

GALENA incorporates various techniques for locating the critical failure surface with user-supplied restraints. Back analyses can also be performed to obtain critical material strength parameters from known or assumed failure surfaces, and probability analyses performed to gauge the likelihood of Factors of Safety being below values of interest, based on expected material property variations.

Either effective or total stresses may be used on any material layer. For the total stress case, the increase in undrained shear strength with depth can be simulated using Skempton's relationship by simply entering the value of the plasticity index for that material.

Probabilistic analysis can be readily undertaken using either defined material properties, or defined mean values, and standard deviation for the production of density and distribution plots.

GALENA allows shear strength to be defined using traditional c and phi values, the Hoek-Brown (1983) failure criterion (m, s and UCS), or with shear/normal data from curves of any shape.

2.9.1 Methods of Analysis

GALENA incorporates three different methods of slope stability analysis. These are:

i. BISHOP SIMPLIFIED METHOD - suitable for circular failure surfaces.
ii. SPENCER-WRIGHT METHOD - suitable for circular and non-circular failure surfaces.
iii. SARMA METHOD - suitable for more complex problems particularly where non-vertical slice boundaries (such as faults or discontinuities) are significant.

In most instances, slope stability problems can be analyzed with one of the above methods. However, for complex slope stability problems where in-situ stresses are significant, it may be more appropriate to use a stress analysis method such as finite element or finite difference etc. Nevertheless, ***GALENA*** will provide rapid answers for most slope stability problems and it has some features that are designed specifically for the practicing geotechnical engineer, which are detailed within this Users' Guide.

GALENA calculates factor of safety through a surface which is most likely to fail in comparison to other surfaces adjacent to it. It can also produce another 99 failure surfaces which are likely to fail after this.

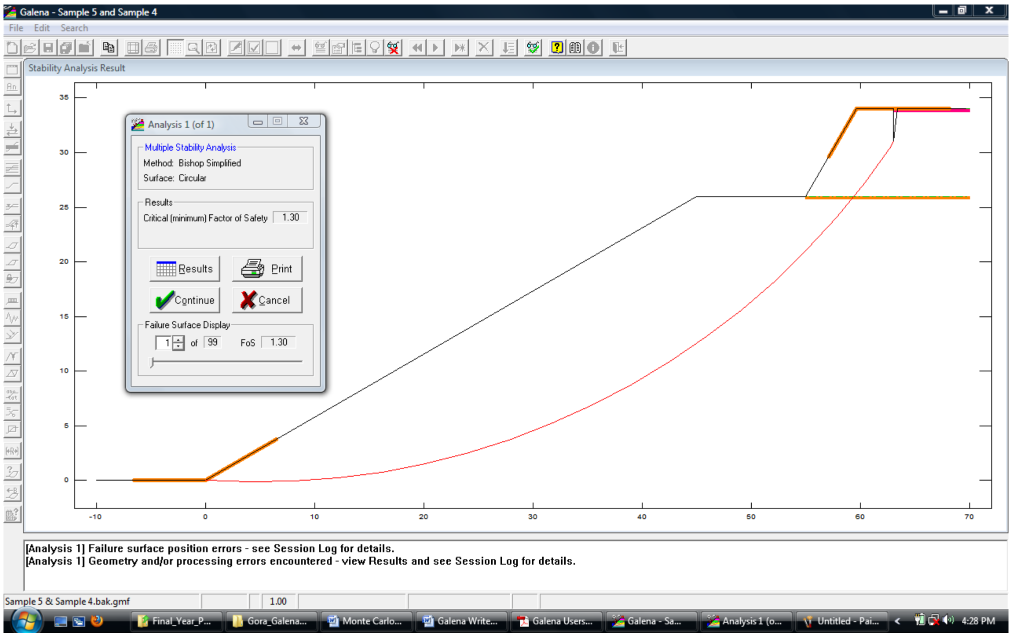


Fig 2.10: A program showing one critical failure surface through a slope

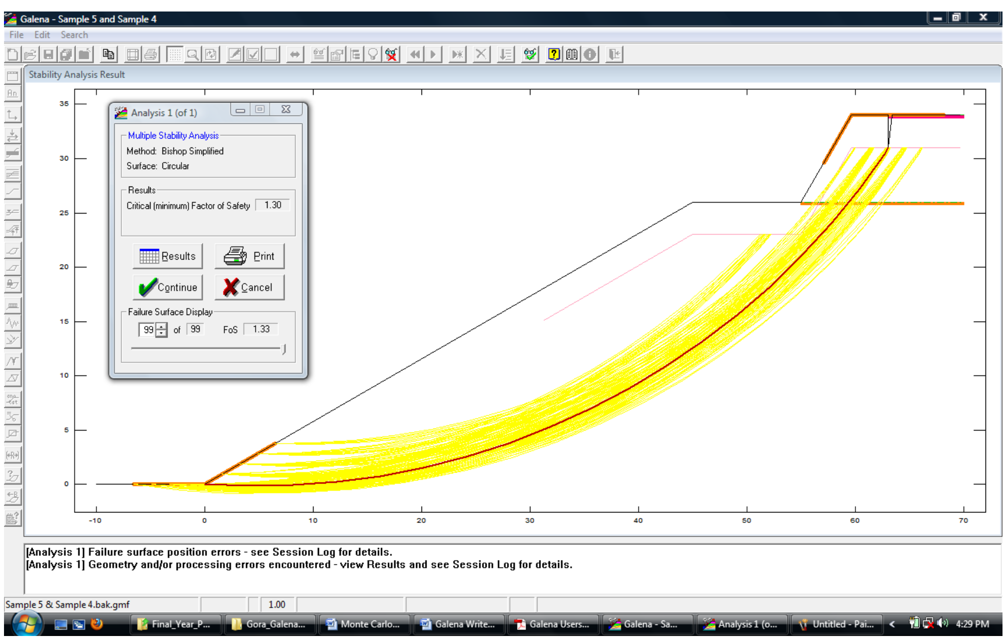


Fig 2.11: A program showing 99 critical failure surfaces through a slope.

2.9.2 Numerical Errors - Limit Equilibrium Methods of Analysis

For limit equilibrium analysis procedures, numerical errors are known to be associated with the following cases:

a) Cohesive soil slopes with a shallow failure surface or where a high cohesive layer exists along the upper portion of the failure surface. Negative stresses may be generated towards the top of the failure surface.

b) Where a steeply dipping section of a circular surface is present in the toe region, particularly when a relatively thin cohesion-less layer overlies a thicker layer of weak clay. Similar problems may be encountered with non-circular failure surfaces where a sub-horizontal surface is present at shallow depth and connected to the ground (slope) surface by a steeply inclined section. Very large or negative stresses may result under these conditions.

CHAPTER – 3

EXPERIMENTAL TECHNIQUES AND RESULTS

3.1 General Description of the Mine

The aim of the study is to investigate the existing status of slope of the mines. So, a nearby operating mine was considered for investigation. Though the state Odisha is mineral rich, deposits are mostly concentrated in the district of Keonjhar.

The region Keonjhar has a no of mines; particularly surface mines as a host of minerals such as Iron, Manganese, Dolomite, etc. Many of these mines have private ownerships and they hold limited lease area to operate the mine. The mine under investigation belongs to Joda, Keonjhar District, Odisha which is about 141 miles from Bhubaneswar, the capital city of the state in the eastern region of India, well connected by road to Keonjhar (80 Km) to Barbil (35 Km) and Jamshedpur (180 Km). The production capacity of the mine is 2 million MT. The typical ores in this region are Haematite, Magnetite, Goethite and Siderite. The major chemical composition of the iron ore produced here are Haematite (Fe_2O_3), Magnetite (Fe_3O_4). The cut-off grade of Iron in the ore is 55%. Any materials containing less than 55% of iron are considered as waste materials.

3.1.1 Mining

The method of working is opencast mechanized mining considering various technical parameters like surface topography, continuation of iron deposit, quality variations, geo-technical aspects, required rate of production etc. The deposit is mined by adopting 10.0 m bench height and with width more than the height of benches i.e. more than 10 m, with an ultimate pit slope of 45°. The transportation of the materials is done using Shovel-Dumper combination.

3.1.2 Transportation

The blasted materials are transported using shovel-dumper combination. The same is the case for waste materials. Dumpers of 25T and 35T capacity are used for this very purpose.

3.1.3 Dumping

The waste generated in the years of the plan period was accommodated in a waste dump situated in the Western part of the mine. The dump is situated to be more or less 1 km from the mine. Area available for dumping is 375m × 225m. The waste materials are dumped with the use of dumper capacity of 25 Tons. The materials are compacted through the conventional mechanism. The present height of the dump is about 32m.

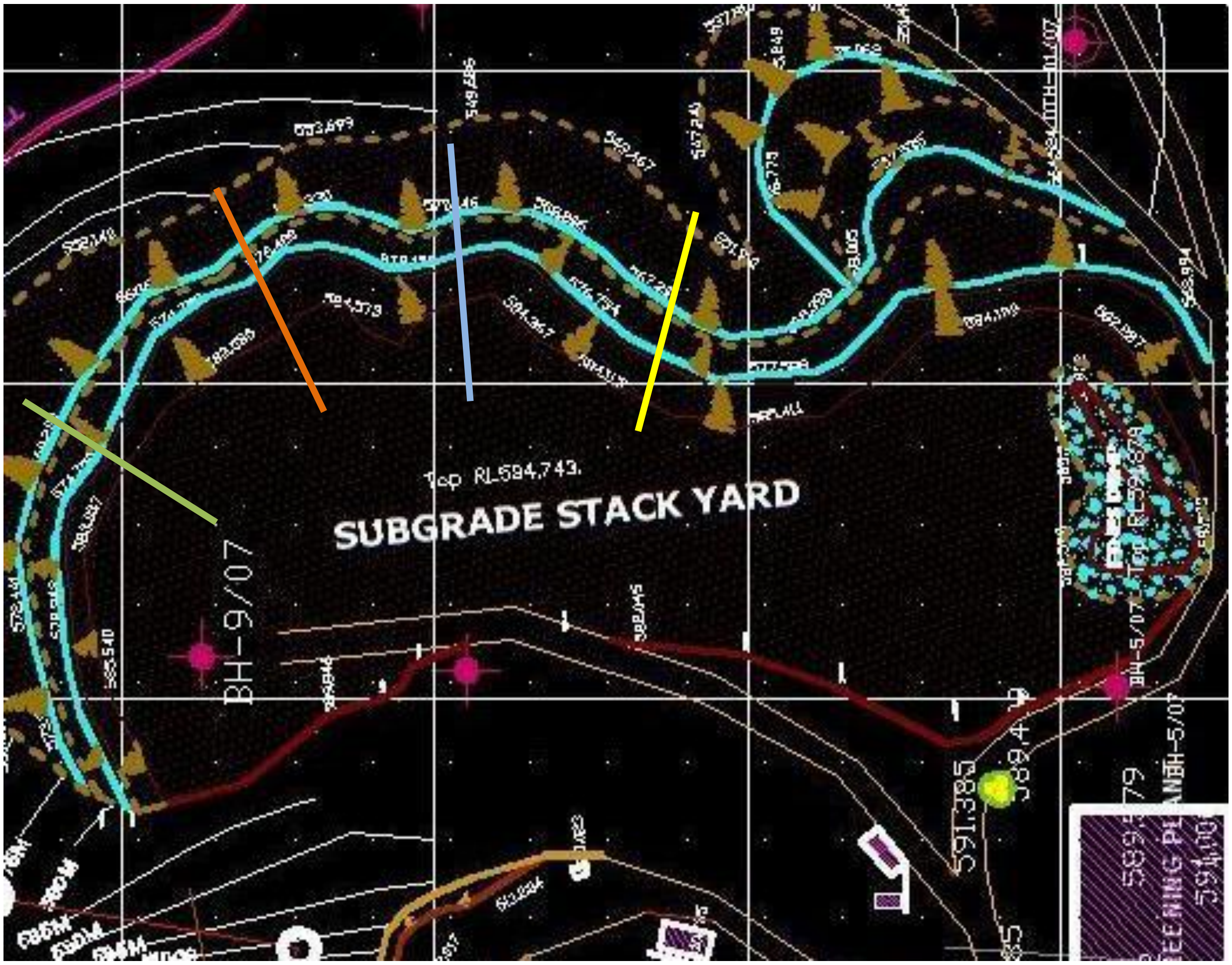


Fig 3.1: Plan of the Waste dump of the mine under study

Section AA'

Section BB'

Section CC'

Section DD'

The plan of the waste dump of the mine is shown in the fig 3.1**.** As no mineral processing plant is present, the sub-grade materials are stacked in the form of a dump, which is also known as a sub-grade stack yard.

The dumpers usually forge the load and dump at the central position of the area and move towards the boundary. Dozers then spread and level the waste.

3.2 Field Observations

The investigation involves determination of geo-technical parameters, study of mine plans, dumping process, etc. So to collect data, a no of visits were incurred to the mine including dump site and the following observations are drawn.

- There were wide variations in the size of materials dumped.
- One portion of the dump (section CC') has failed. This is because of the apparent seepage of rain water through this area. The Fig 3.2 and 3.3 reflects the respective observation

Fig 3.2: Slope failure at the section CC'

Fig 3.3: The top of the section CC' under failure.

3.3 Sample Collection and Preparation:

The samples from the area were so collected that they represented the area under study. The samples were obtained from eight different locations at the time of winter. The samples were taken at about depth of 2-3 feet from the surface of the dumps. Firstly, a cylindrical structure was hammered into the surface up to the stipulated depth and then the container with soil sample was carefully taken out, immediately packed into a plastic gunny bag so as to avoid interference of atmospheric conditions and brought to the laboratory. It was ensured that the sample preserved its virgin state.

Fig 3.4: Site Preparation.

Fig 3.5: Hammering of the mould into the hole

Fig 3.6: Removal of the mould.

Fig 3.7: Picture of the place after sampling.

Then the samples were sieved to required sizes (3.75mm) and stored in air tight polythene packets. The packets were stored in air tight containers for further use in experimentation.

3.4 Experimental Methods:

The Geotechnical parameters needed for the stability analysis are:

1. Unit weight 'γ'.
2. Cohesion 'c'
3. Friction angle 'ϕ' (UU test)
4. Angle of repose 'ß'
5. Pore water pressure.

But because of the unavailability of the instruments in the laboratory, only Proctor compaction test and Tri-axial test of the samples were carried out.

3.4.1 Proctor Compaction Test (ASTM D698):

Aim: To determine the Maximum dry density and optimum moisture content.

Equipment

- Proctor mould with a detachable collar assembly and base plate.
- Manual rammer weighing 2.5 kg and equipped to provide a height of drop to a free fall of 30 cm.
- Sample Extruder & A sensitive balance.
- Straight edge.
- Squeeze bottle
- Mixing tools such as mixing pan, spoon, trowel, spatula etc.
- Moisture cans.
- Drying Oven

Fig 3.8: Proctor Compaction Apparatus

Fig 3.9: Application of blows.

Table 3.1: Results of Proctor compaction Test

Test #	1	2	3	4
Weight of the mold without the base and collar, W_1, (Kg)	3.736	3.736	3.736	3.736
Weight of the mold + moist soil, W_2 (Kg)	6.524	6.743	6.693	6.688
Weight of the moist soil, $W_2 - W_1$, (Kg)	2.784	3.003	2.953	2.948
Moist unit weight, $\gamma_t = \frac{W_2 - W_1}{1000} \times 1000$ (gm/cc)	2.784	3.003	2.953	2.948
Moist unit weight in (Kg/m^3)	2784	3003	2953	2948
Weight of moisture can, W_3, (g)	12.4	21.23	20.32	21.1
Mass of can + moist soil, W_4, (g)	85.68	110.19	179.34	229.86
Mass of can + dry soil, W_5, (g)	80.3	101.65	159.8	196.47
Moisture content: $w(\%) = [(W_4 - W_5)/(W_5 - W_3)] \times 100$	7.923	10.62	14.01	16.76
Dry unit weight of compaction: $\gamma_d = \gamma_t / [1+(w/100)]$	2580	2715	2590	2525

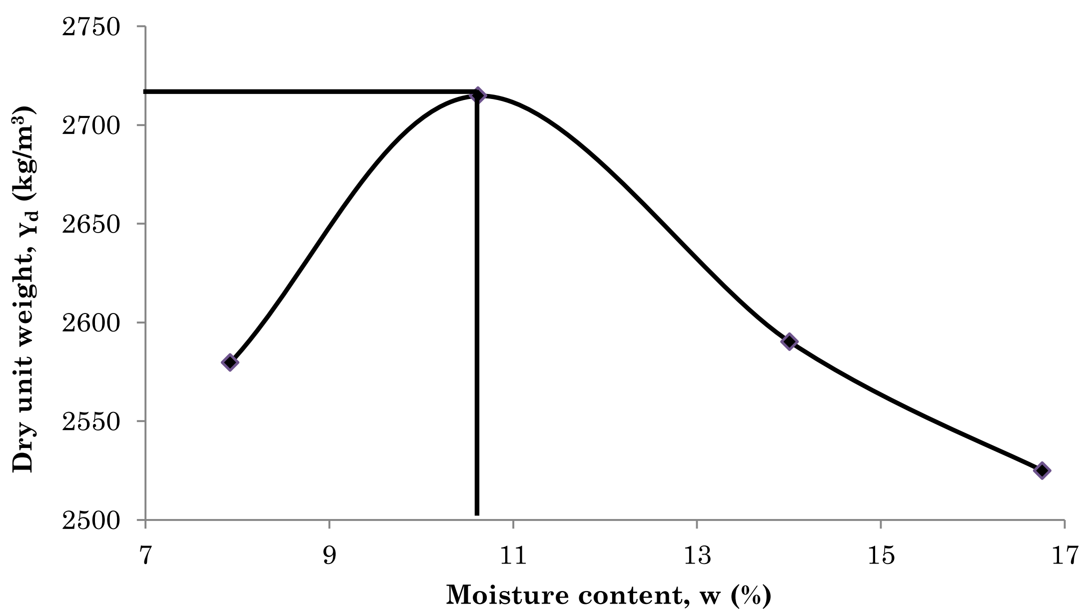


Fig 3.10: Relation between Dry unit weight and Moisture Content.

From the graph, it can be found that:

Maximum Dry Density = 2715 kg/m^3

Optimum moisture content = 10.62%

3.4.2 Tri-Axial Test (ASTM D2850):

This test method covers determination of the strength and stress-strain relationships of a cylindrical specimen of either undisturbed or remolded cohesive soil. Specimens are subjected to a confining fluid pressure in a triaxial chamber. No drainage of the specimen is permitted during the test. The specimen is sheared in compression without drainage at a constant rate of axial deformation (strain controlled).

This test method provides data for determining undrained strength properties and stress-strain relations for soils. This test method provides for the measurement of the total stresses applied to the specimen, that is, the stresses are not corrected for pore-water pressure.

Apparatus

For conducting the test, the testing system consists of the following five major functional components:

a) A system to house the sample, that is, a triaxial cell;
b) A system to apply cell pressure and maintain it at a constant magnitude;
c) A system to apply additional axial stress;
d) A system to measure pore water pressure; and
e) A system to measure changes of volume of the soil sample.

Elements Used within the Triaxial Cell

The Tri-axial Test may be programmed so as to allow or exclude the hydraulic connection between the inside of the sample with the ambient outside the tri-axial cell or with special measuring instruments. Such connections might require the use of special drainage mediums around the sample, in particular: Porous Discs on the top and bottom of the sample and Filter Drains around its sides. However, when the sample has to be isolated, the bottom porous disc has to be replaced by an impermeable Base Disc whilst the upper porous disc is removed. In each case the sample is placed on a Pedestal and a Top Cap is placed on top of the sample. These elements will have the same diameter as the sample. To make the sample isolated from the water within the triaxial cell, it is covered with a very thin Membrane made of natural rubber (of appropriate diameter) which is placed over the sample using a Suction Membrane Stretcher and a water-tight fit is guaranteed at the junction with the pedestal and top cap by using Sealing Rings of appropriate diameter.

Fig 3.11: The tri-axial testing apparatus used for this project.

Fig 3.12: The Tri-axial Cell during testing process

3.4.2.1: Sample Preparation for Tri-axial Testing

The samples are prepared with the use of a cylindrical mold (Fig 3.13). It has the following specifications.

Length (cm)	Diameter (cm)	L/D ratio	Volume (cm^3)
7.6	3.8cm	2	86.2

As calculated from proctor compaction Test,

Maximum Dry Density =2715 kg/m^3

Optimum moisture content =10.62%

Hence, Mass of the sample needed = 2.715 × 86.2 = 234.033 gm.

Water required = $\frac{234.033 \times 10.62}{100} = 24.85$ ml.

Fig 3.13: The cylinder used for the preparation of sample.

Fig 3.14: Sample of size < 3.75mm

Fig 3.15: Preparation of the sample

Fig 3.16: Cylindrical sample before Test

Fig 3.17: Sample after the test

The cylindrical shaped samples were tested using Tri-axial apparatus. A stress vs. strain curve was plotted. The peak value of the stress is considered as the deviatory stress ($\Delta\sigma$), from which the corresponding major and minor principal stresses are found out.

Minor Principal Stress (σ_1) = Cell confining Pressure ($\Delta\sigma$).

Major Principal Stress (σ_3) = Deviatory Stress (Calculated from the stress-strain curve of the triaxial test) + Minor Principal Stress.

$$\sigma_3 = \sigma_1 + \Delta\sigma$$

Table 3.2: Result from the Tri-axial Test

Sample No	Deviatory Stress ($\Delta\sigma$)	Minor Principal Stress (σ_1), kPa	Major Principal Stress ($\sigma_3 = \sigma_1 + \Delta\sigma$), kPa
1	136	100	236
	216	200	416
	247	300	547
2	129	100	229
	190	200	390
	239	300	539
3	112	100	212
	156	200	356
	213	300	513
4	108	100	208
	159	200	359
	198	300	498
5	215	100	315
	378	200	578
	533	300	833
6	108	100	208
	173	200	373
	250	300	550

7	188	100	288
	380	200	580
	512	300	812
8	236	100	336
	389	200	589
	501	300	801

From the values of the Major and Minor principal stress, the cohesion(c) and friction angle (ϕ) values are calculated using Mohr-Coulomb criterion. This was possible with the help of RocData software.

3.5 Mohr Coulomb Analyses

Mohr –Coulomb analysis was carried out by using the program ‘RocData’. Here, the Major Principal Stress and Minor principal stress are given as inputs. The different Mohr’s circles for different samples are shown below.

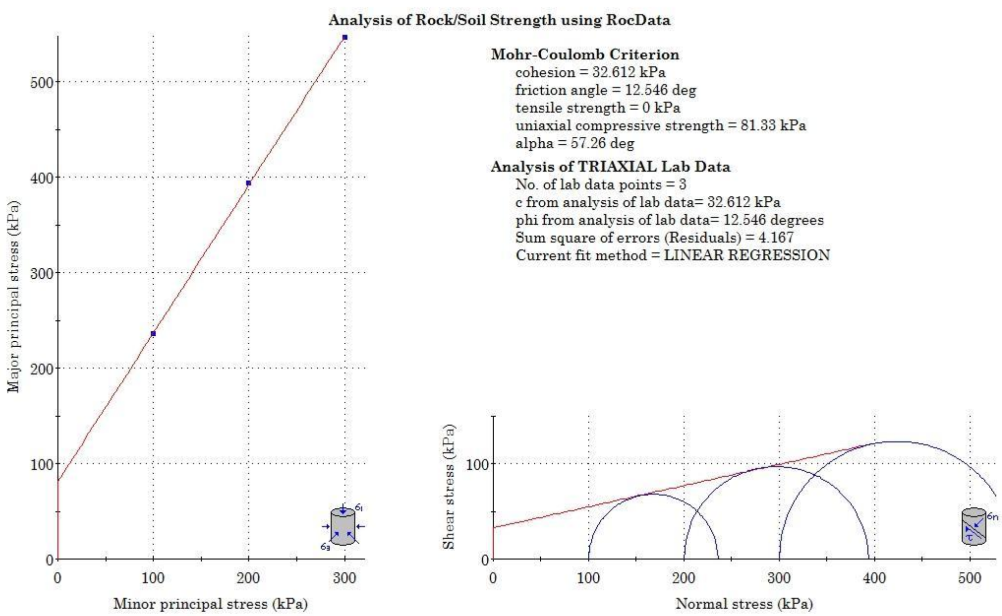


Fig 3.18: Mohr’s circle for Sample 1

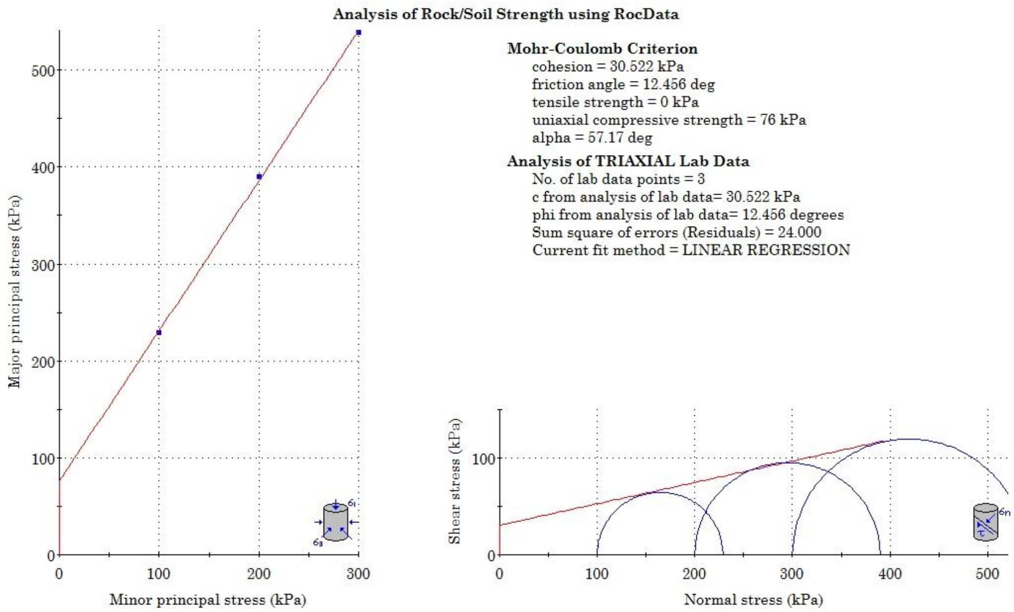


Fig 3.19: Mohr's circle for Sample 2

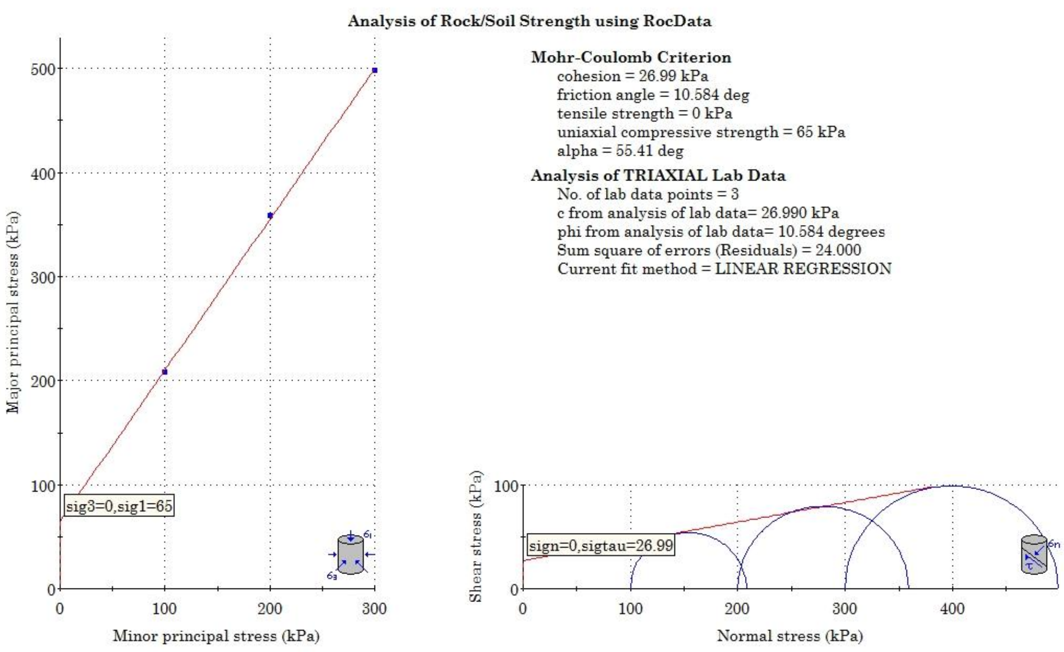


Fig 3.20: Mohr's circle for Sample 3

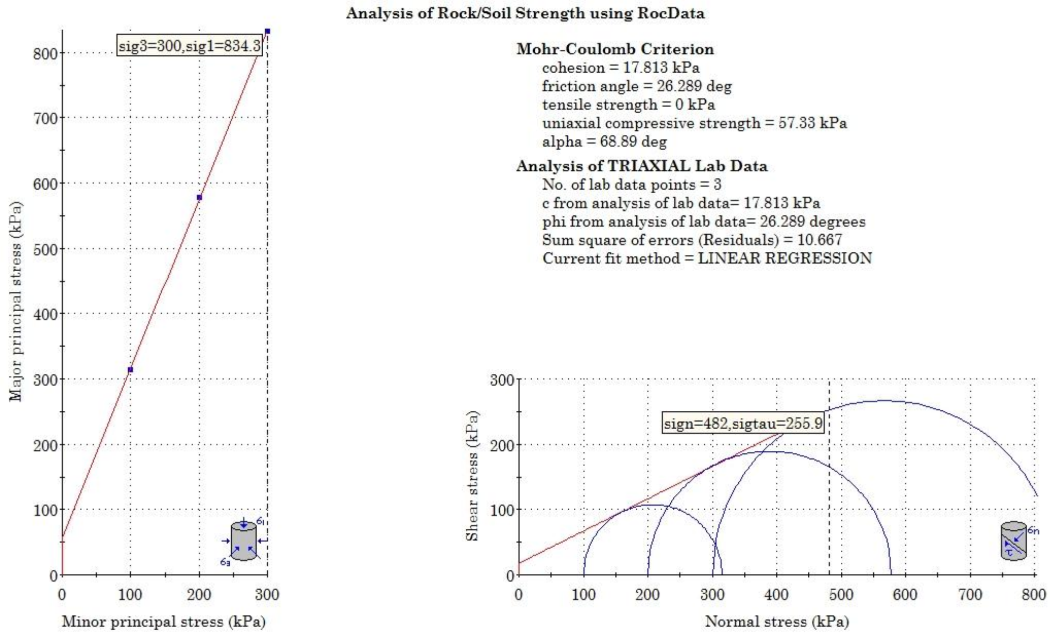


Fig 3.22: Mohr's circle for Sample 5

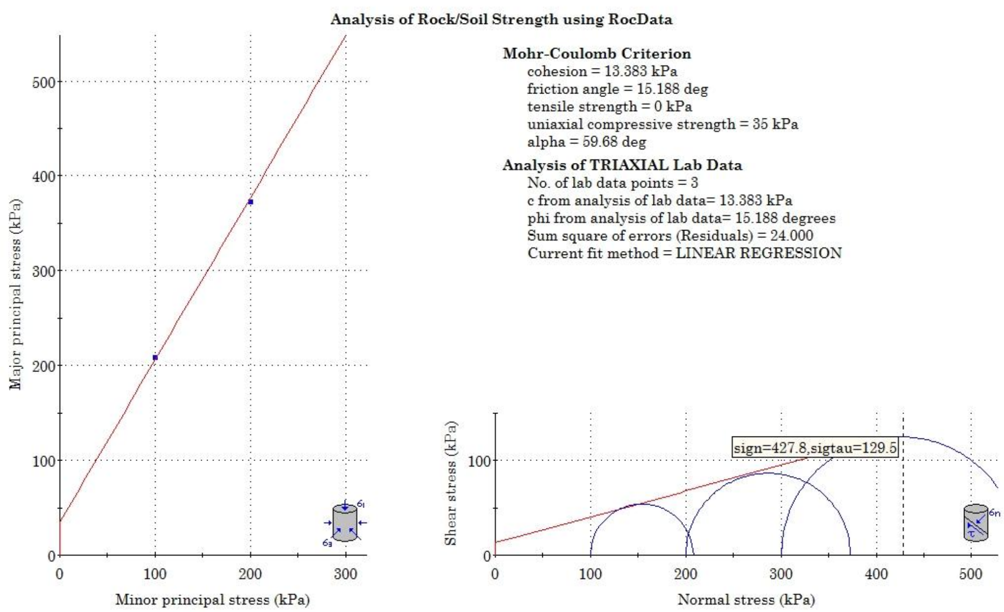


Fig 3.23: Mohr's circle for Sample 6

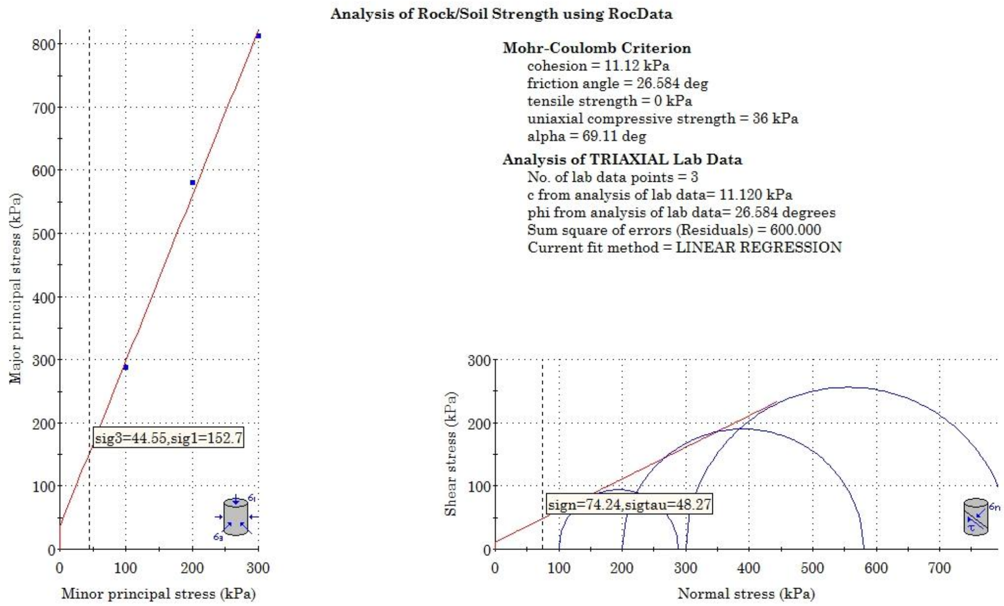


Fig 3.24: Mohr's circle for Sample 7

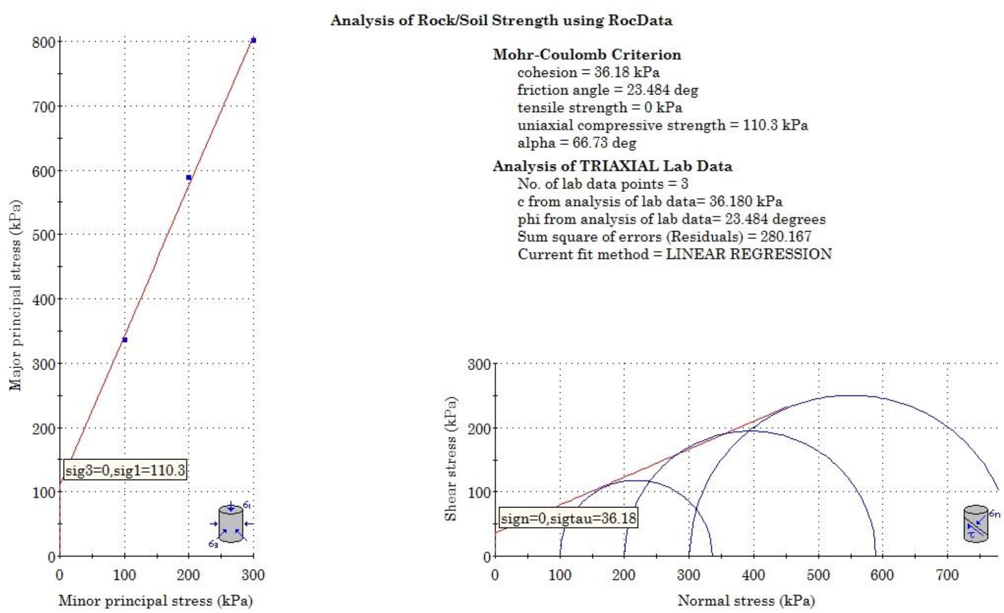


Fig 3.25: Mohr's circle for Sample 8

Table 3.3: Results from the Mohr's circle Analysis

Sample No	Cohesion value in kPa	Friction Angle in degree
1	32.612	12.546
2	30.522	12.456
3	24. 182	11.63
4	26.99	10.584
5	17.813	26.289
6	13.383	15.188
7	11.12	26.584
8	36.18	23.484

Average value

The average value of the cohesion and friction angle is found to be 24.08 kPa and 17.3 degree respectively.

CHAPTER – 4

ANALYSIS

4.1 Valuation of Factor of Safety

The various material parameters of different sections of the dump are tabulated below. From the table 4.1, we can find that the material parameters at different sections represent different values.

Table 4.1: The material parameters of different sections.

Section	Sample No	Cohesion in kPa	Friction Angle in Degree	Density in kN/m^3
AA’	1	32.612	12.546	26.63
	8	36.18	23.484	
BB’	2	30.522	12.456	
	7	11.12	26.584	
CC’	3	24. 182	11.63	
	6	24. 182	11.63	
DD’	4	26.99	10.584	
	5	17.813	26.289	

The density of the materials is 26.63 kN/m^3, which represent the average value of these samples.

Hence, for all those sections shown in the map (Fig 4.1) the inclination of the first slope was considered to be 30^0 and the for the second slope 60^0.

The tension crack was assumed to be situated at 3m from the slope surface to simulate the worst case scenario. Since, multiple restraint analysis was assumed during analysis; factor of safety was not affected much by the position of the tension crack in the slope.

All these sections were modeled in a slope stability analysis system and the existing factor of safety are calculated.

Monte Carlo Simulation was done subsequently to find the reliability index, probability of failure and expected performance level.

4.1.1 Section AA’

Fig 4.1: Design of the section AA’.

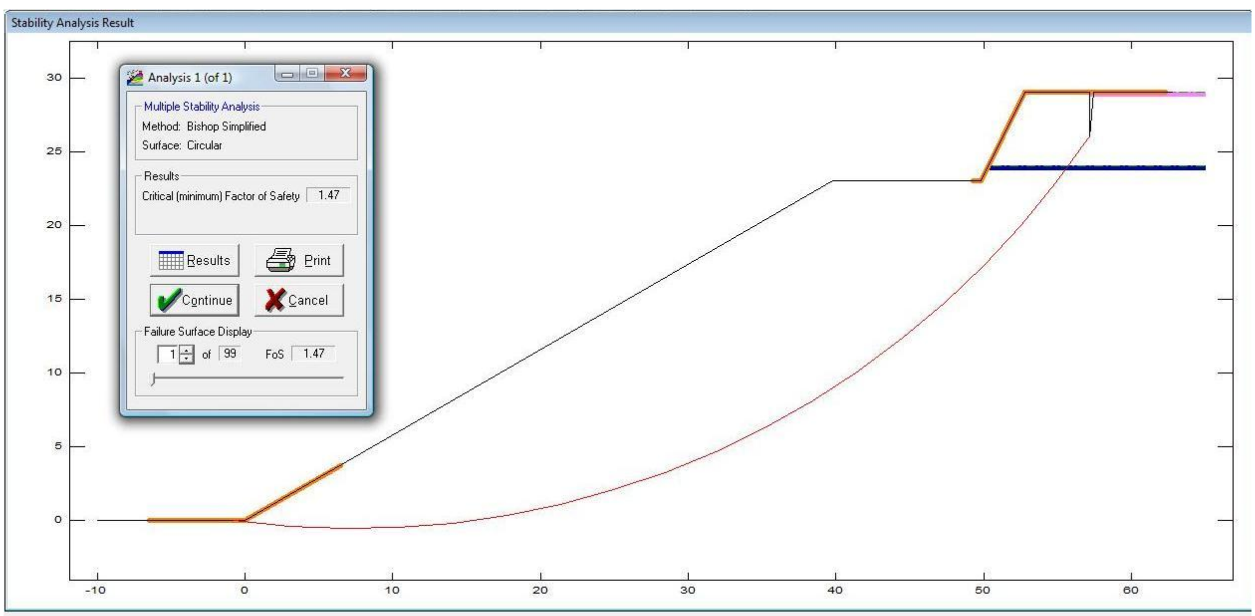


Fig 4.2: The output of the section AA’.

4.1.2 Section BB’

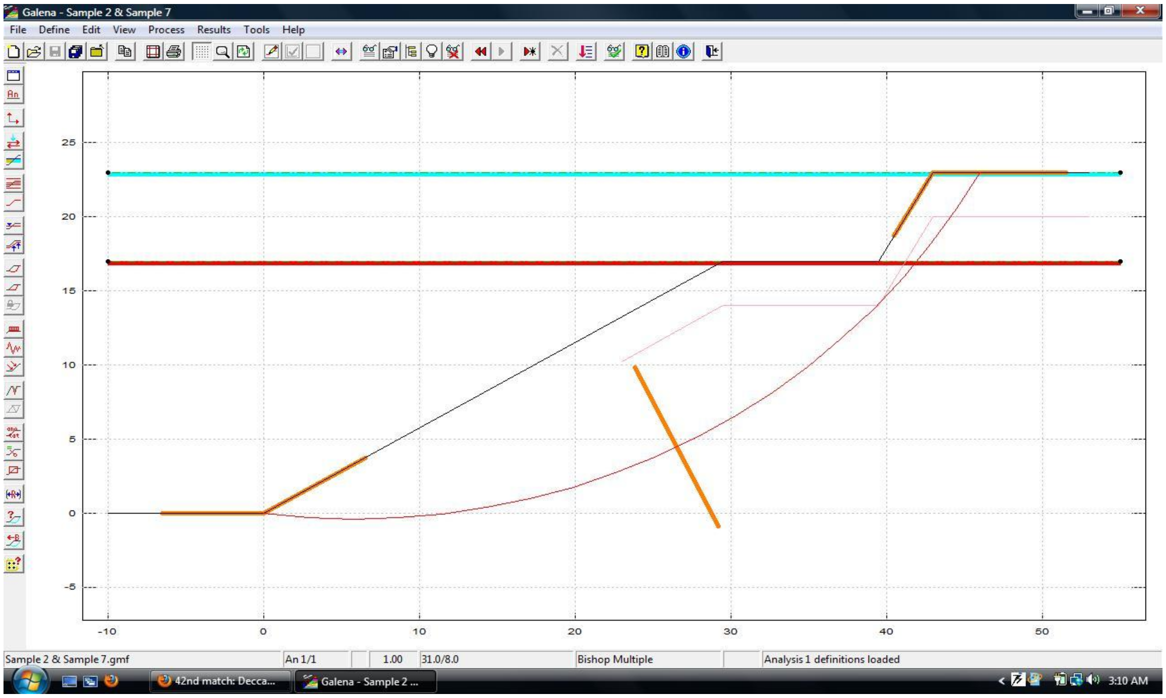


Fig 4.3: Design of the section BB’.

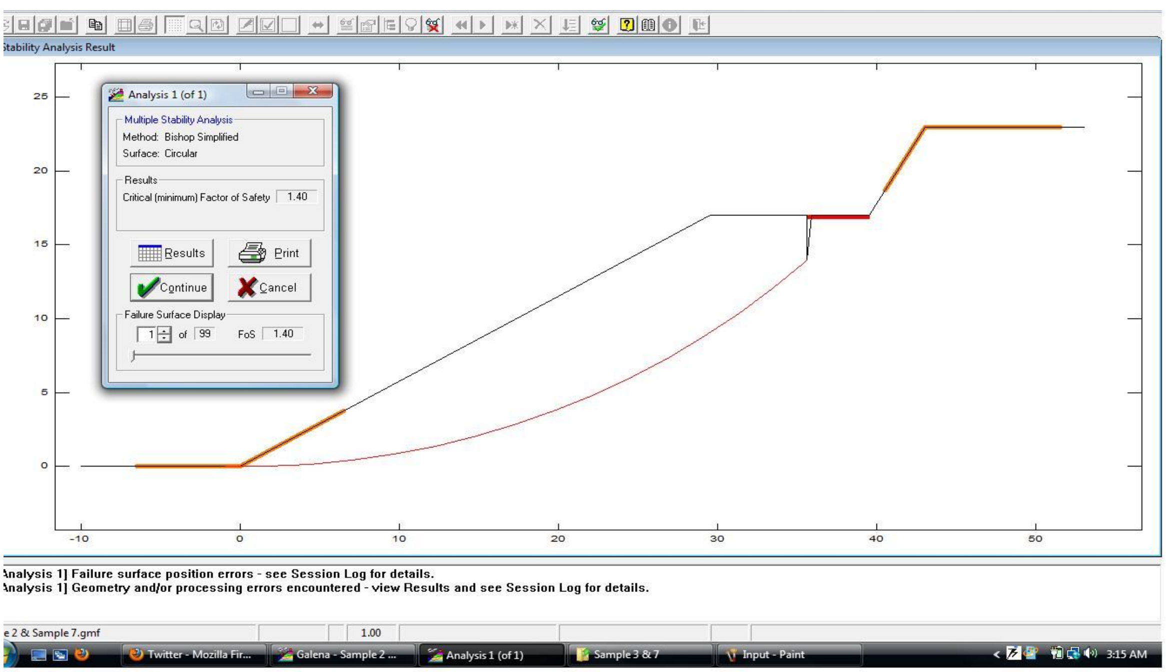


Fig 4.4: The output of the section BB’.

4.1.3 Section CC’

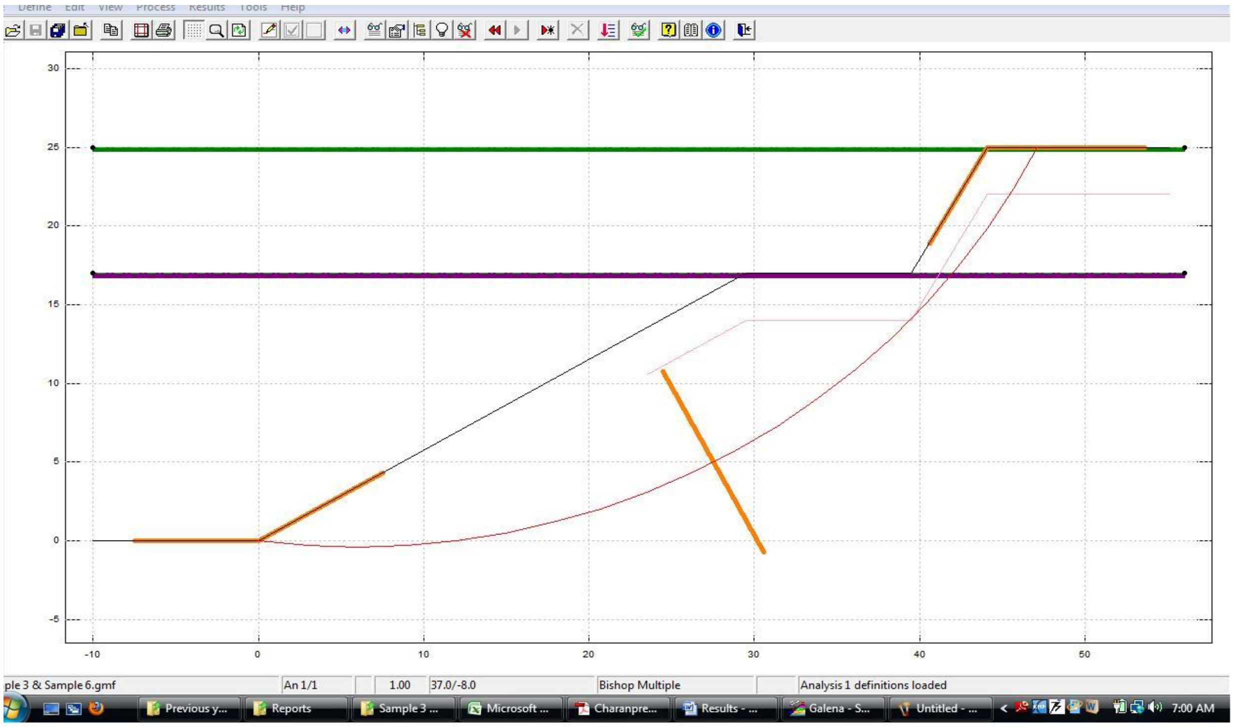


Fig 4.5: Design of the section CC’.

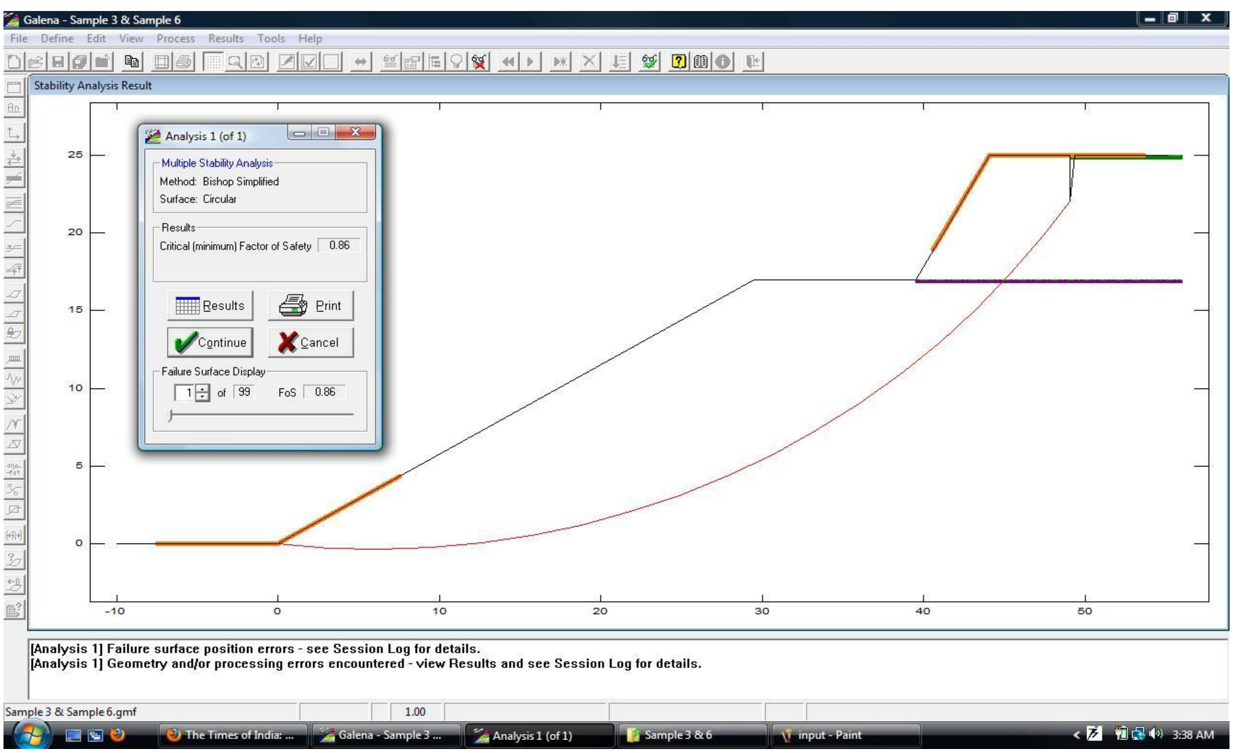


Fig 4.6: The output of the section CC’.

4.1.4 Sample DD':

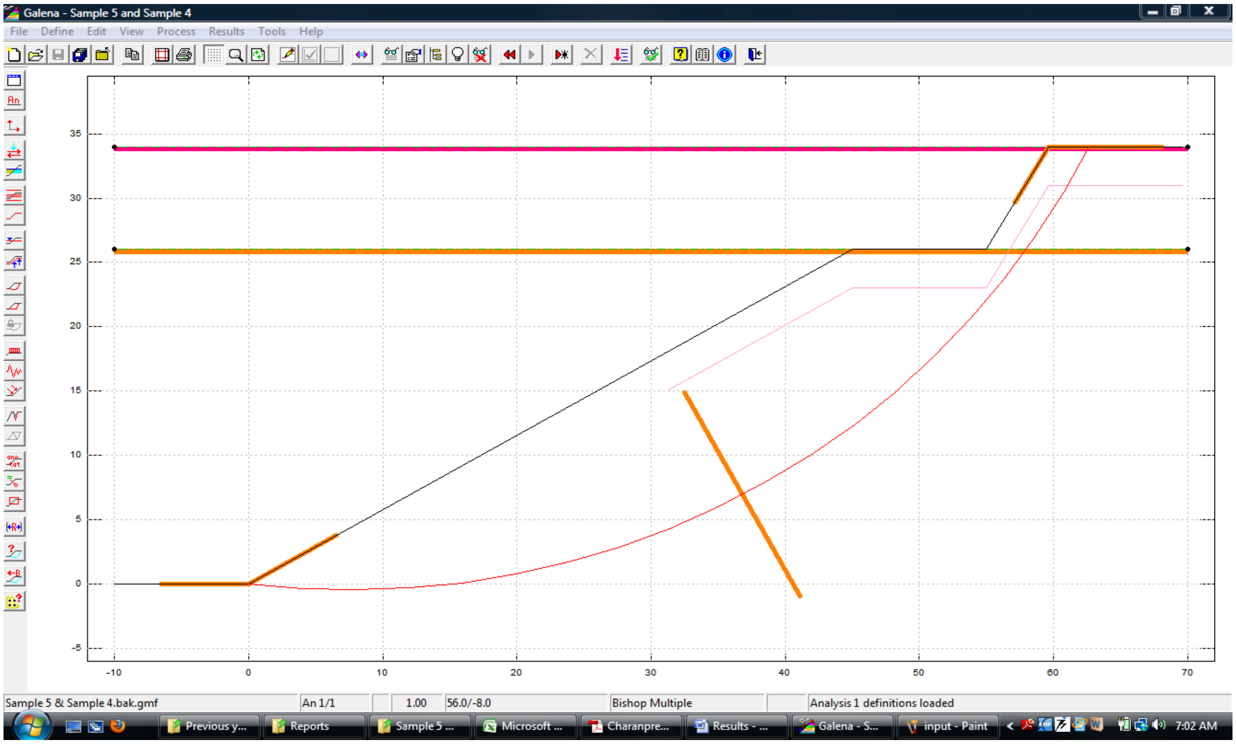


Fig 4.7: Design of the section DD'.

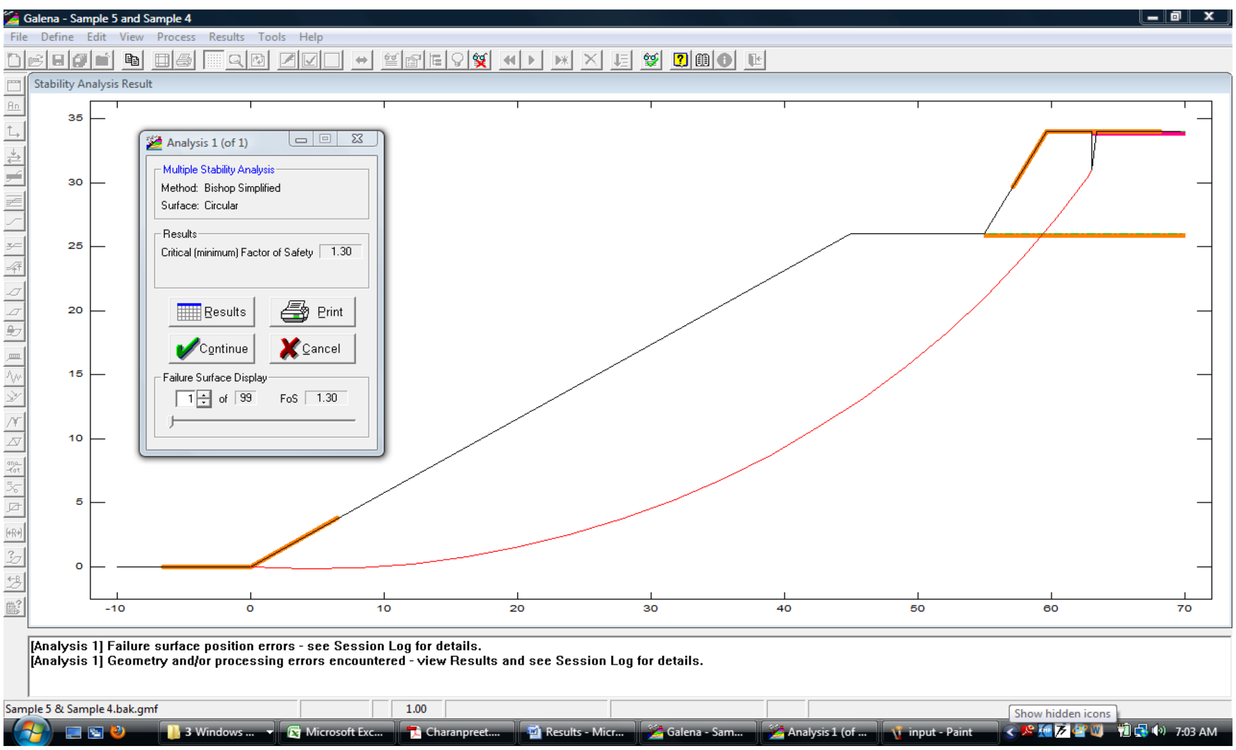


Fig 4.8: The output of the section DD'.

Table 4.2: FOS of different sections of the dump.

Section	Factor of Safety	Comments on the stability
AA’	1.47	Stable
BB’	1.4	Stable
CC’	0.86	Unstable
DD’	1.3	Stable

From the analysis, it is evident that, the slope representing the section CC’ is unstable. Figs (3.2 & 3.3) show the failure of the slope. This is because the percolation of the rain water through the slope.

4.2 Monte Carlo Simulation

The sample no 5, 6, 7 and 8 were collected from the first bench (Bench A) and the sample no 1, 2, 3 and 4 were collected from the second bench (Bench B).

Table 4.3: Characterization of fixed parameters

Slope	**A**	**B**
Bench Height (m)	21	7
Angle of Inclination (degree)	30	60

4.2.1 Material Parameters

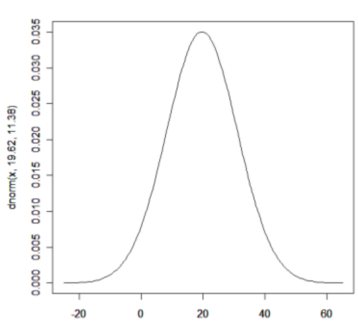
Bench – A:

Mean value of Cohesion = 19.62
Standard deviation = 11.38
Mean value of Angle of Friction = 22.9
Standard deviation = 5.31

Bench – B:

Mean value of Cohesion = 30.04
Standard deviation = 2.84
Mean value of Angle of Friction = 11.9
Standard deviation = 0.911

Bench - A



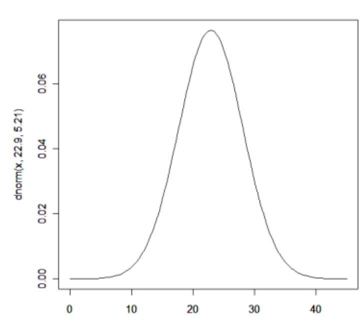


Fig 4.9: Normal Distribution curves for Cohesion and Angle of Friction

Bench - B

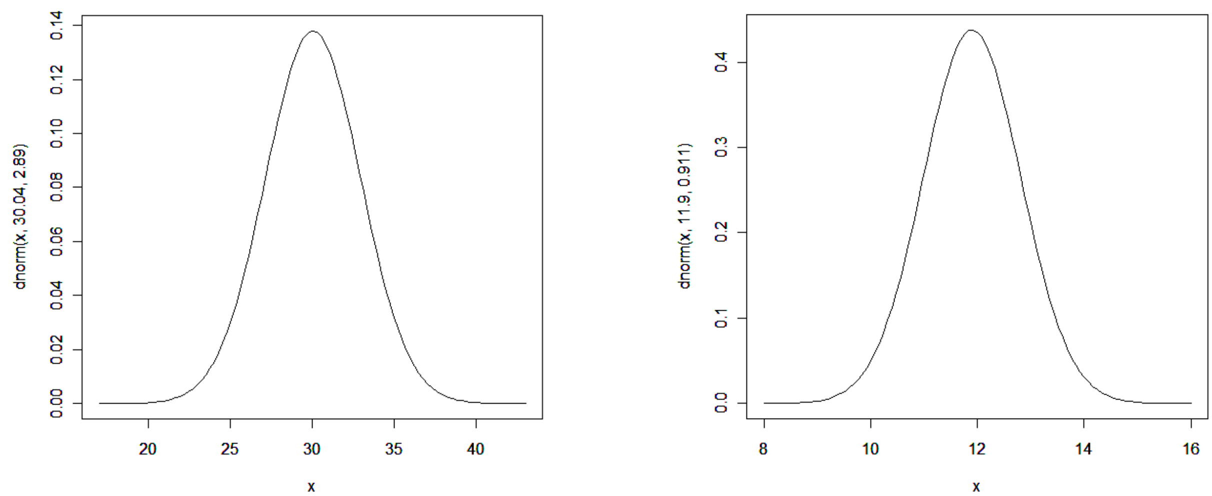


Fig 4.10: Normal Distribution curve for Cohesion and Angle of Friction

In this simulation, the effect of seed random number generator on the reliability index (β) is investigated. For this purpose, several computer runs are conducted by which the seed random number generator is allowed to vary from 100 to 9999.

Two types of Limiting Equilibrium Methods are considered.

- Bishop Simplified Method.
- Spencer Method.

Two types of circular failure surfaces are considered.

- Defined failure surface.
- Critical failure surface.

The figures (4.11 – 4.22) depicts the output of the simulations done by Bishop's simplified method with defined failure surface.

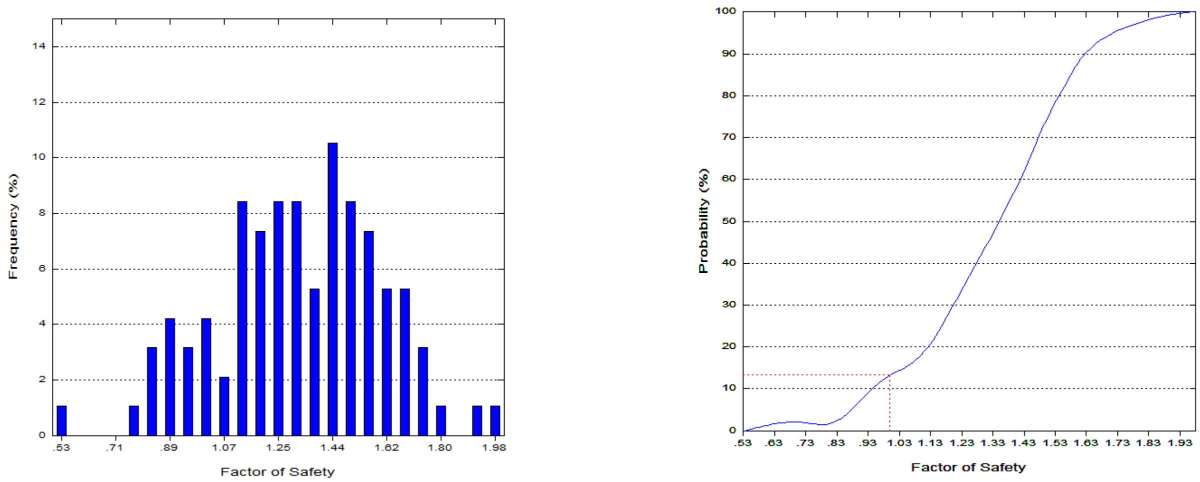


Fig 4.11: MCS with 100 iterations

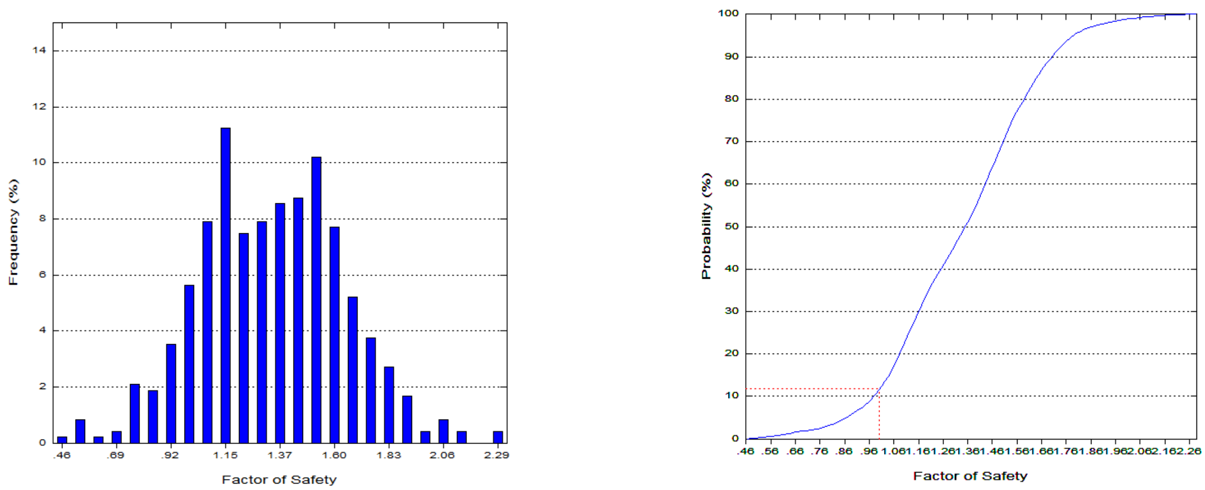


Fig 4.12: MCS with 500 iterations

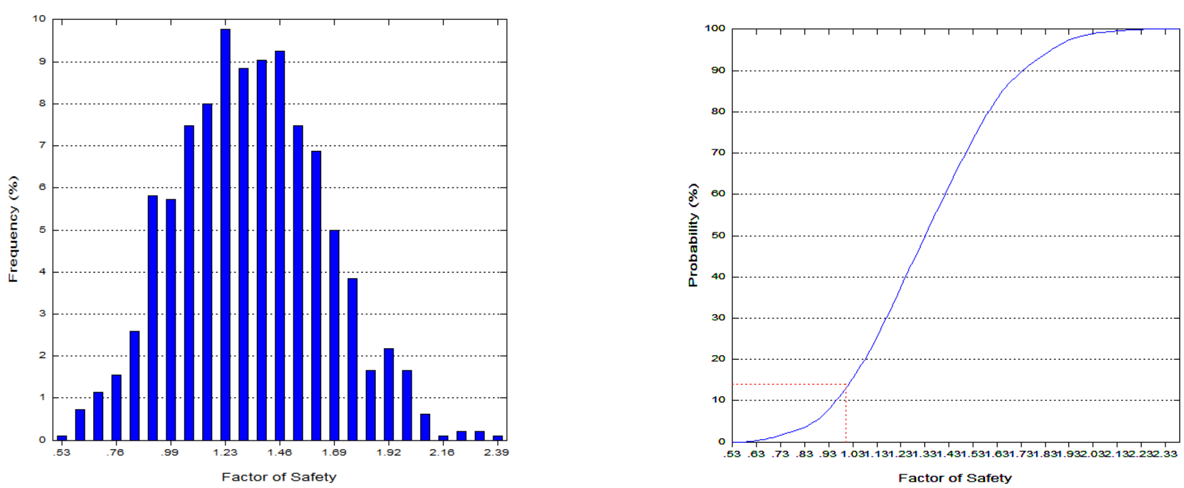


Fig 4.13: MCS with 1000 iterations

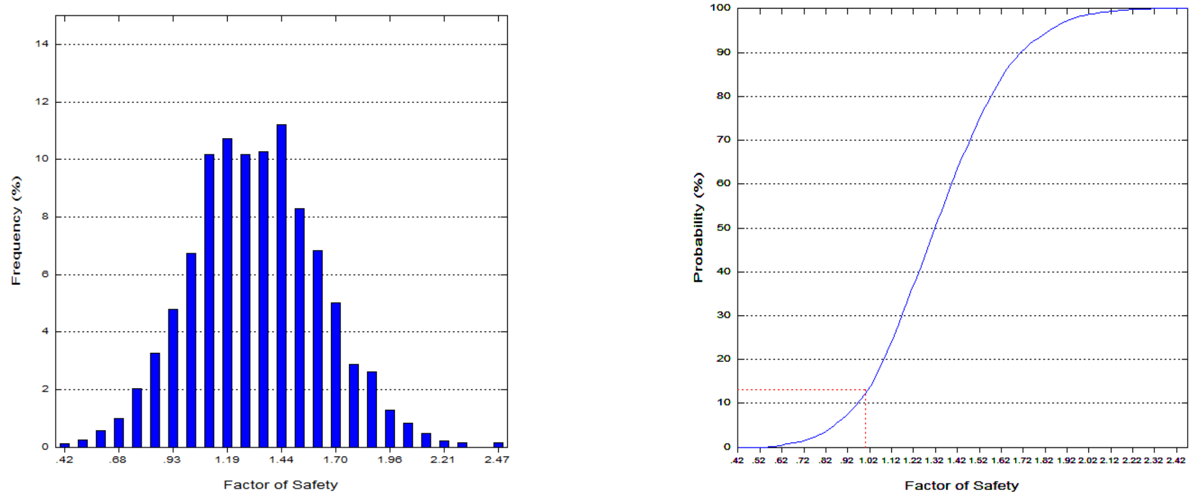


Fig 4.14: MCS with 2000 iterations

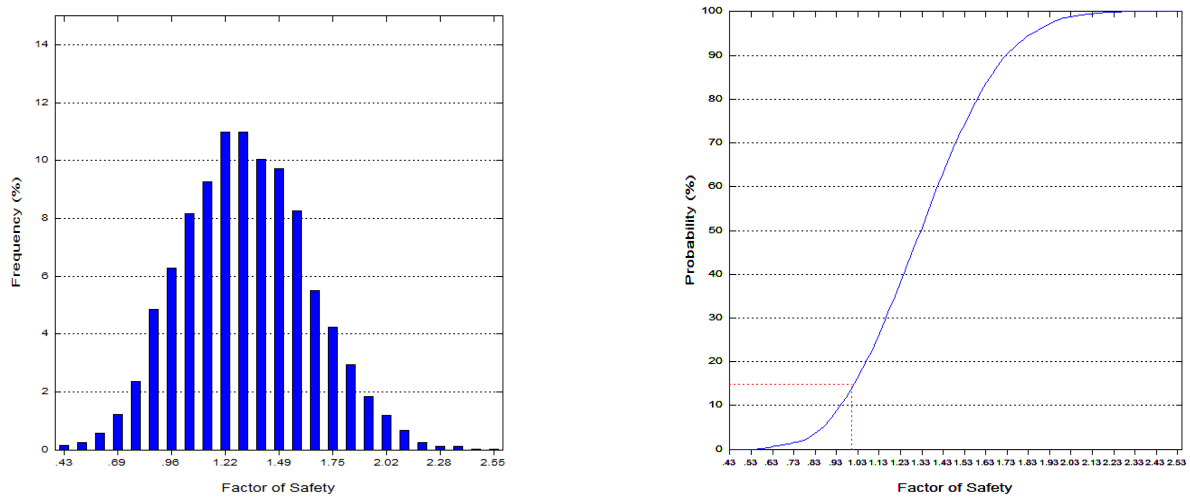


Fig 4.15: MCS with 3000 iterations

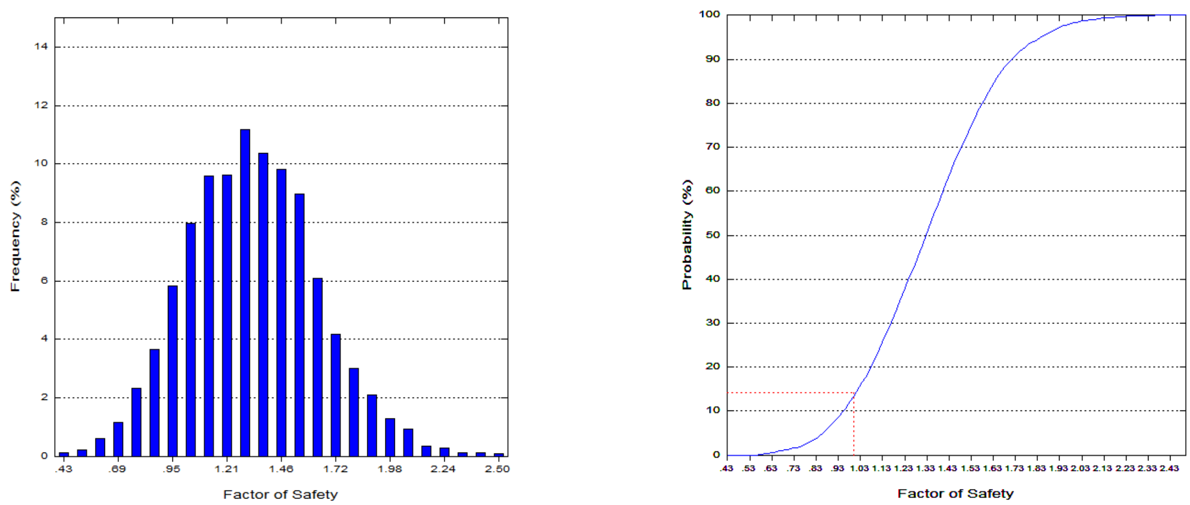


Fig 4.16: MCS with 4000 iterations

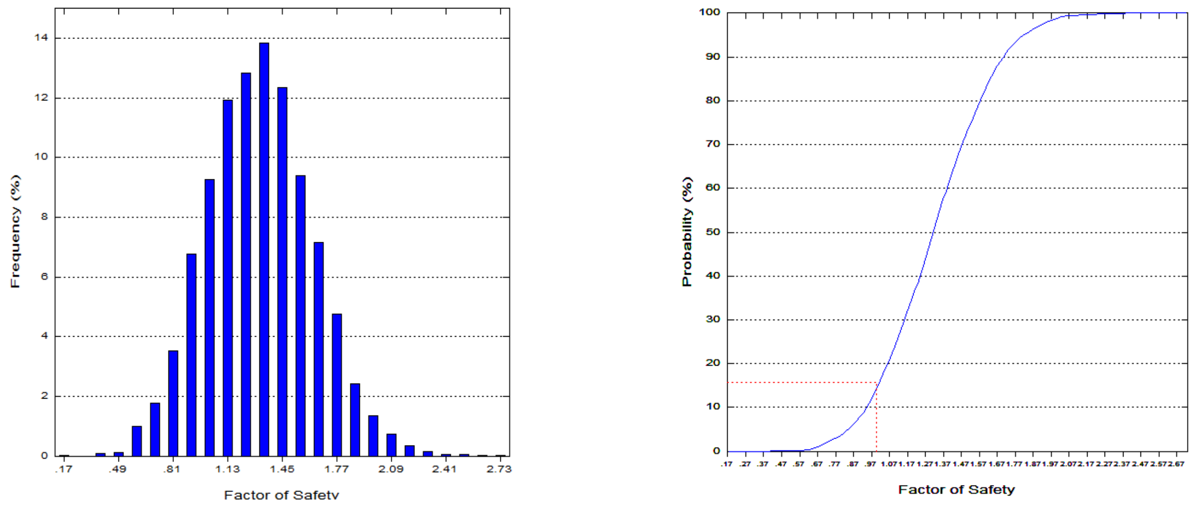


Fig 4.17: MCS with 5000 iterations

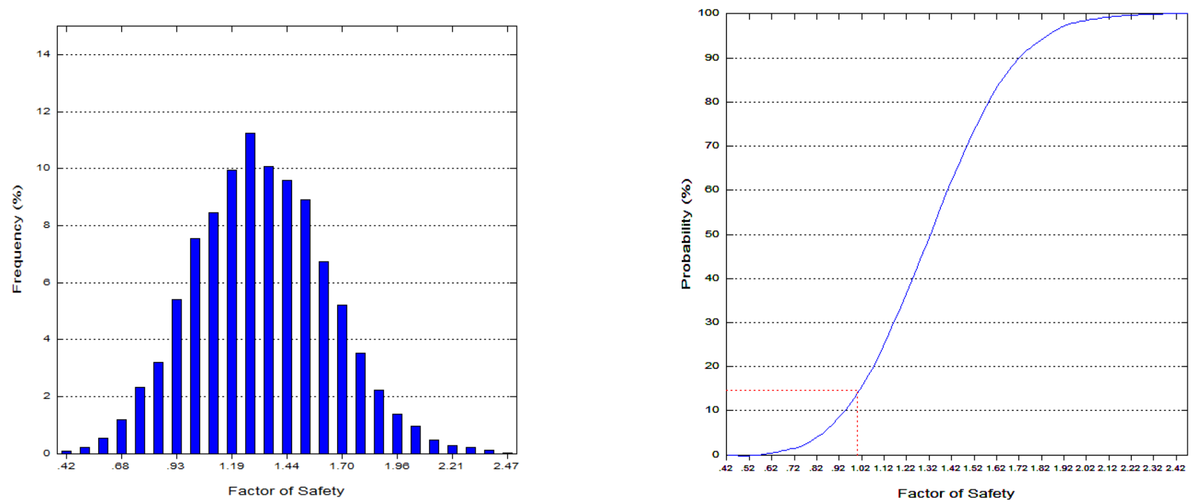


Fig 4.18: MCS with 6000 iterations

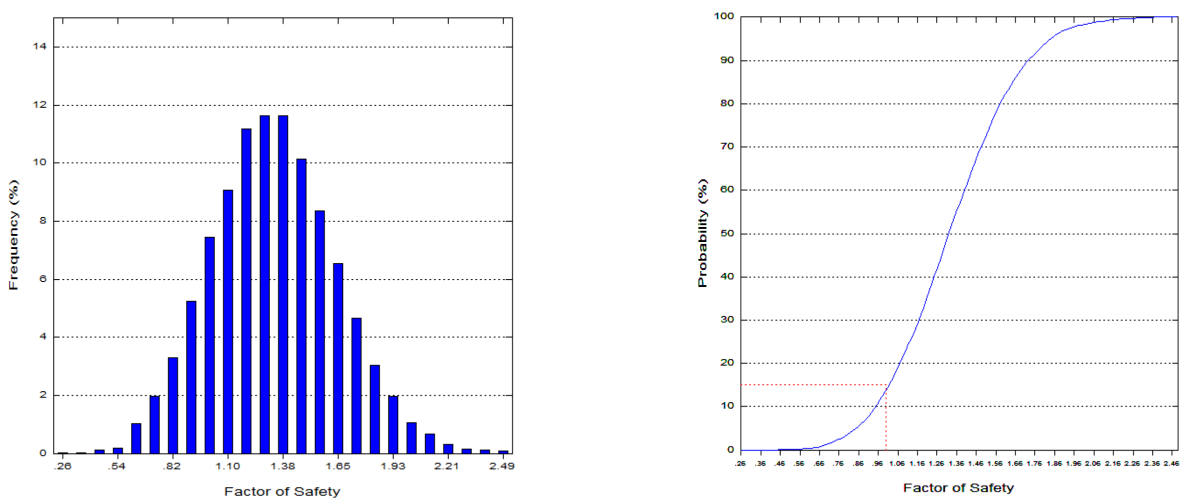


Fig 4.19: MCS with 7000 iterations

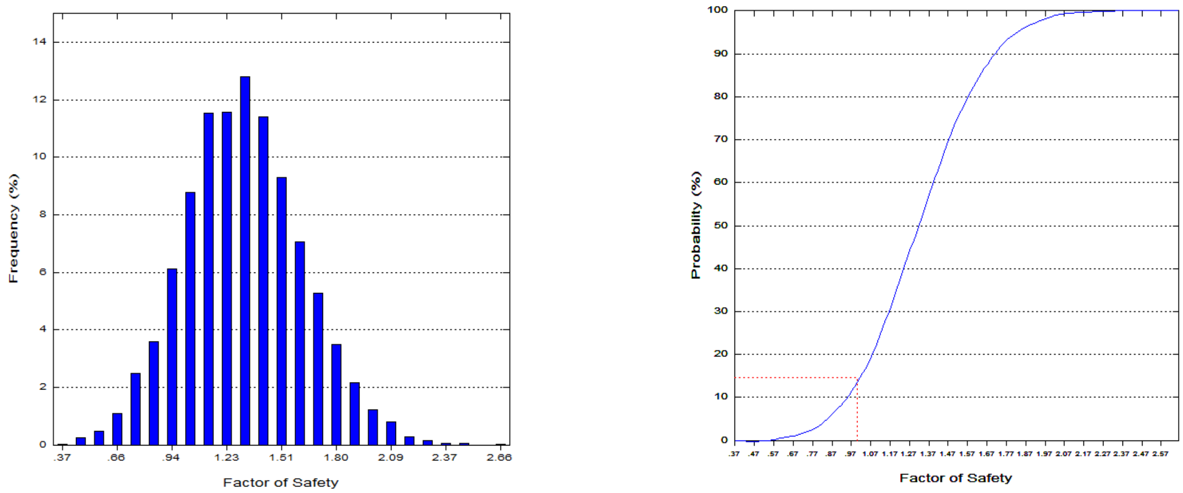


Fig 4.20: MCS with 8000 iterations

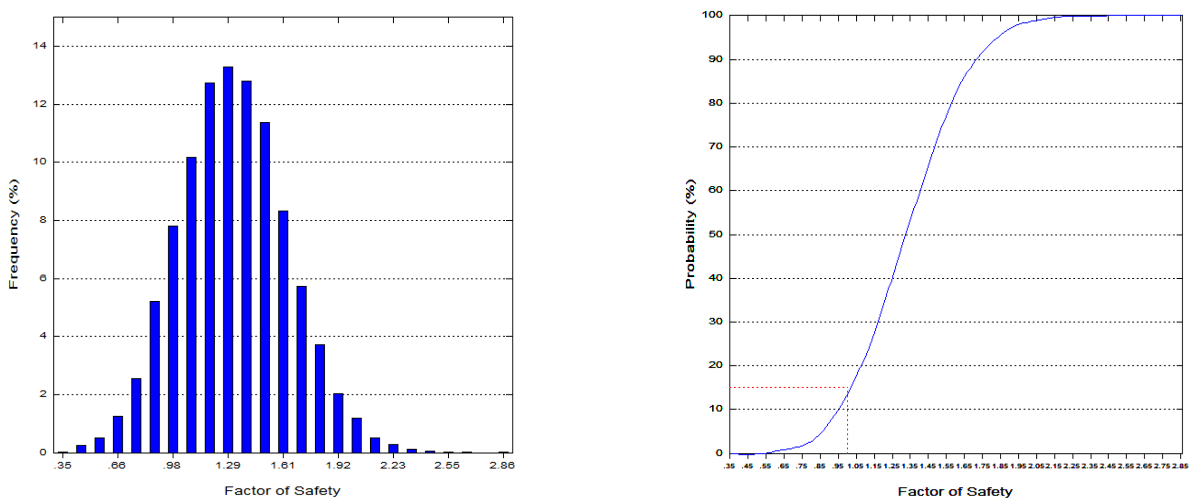


Fig 4.21: MCS with 9000 iterations

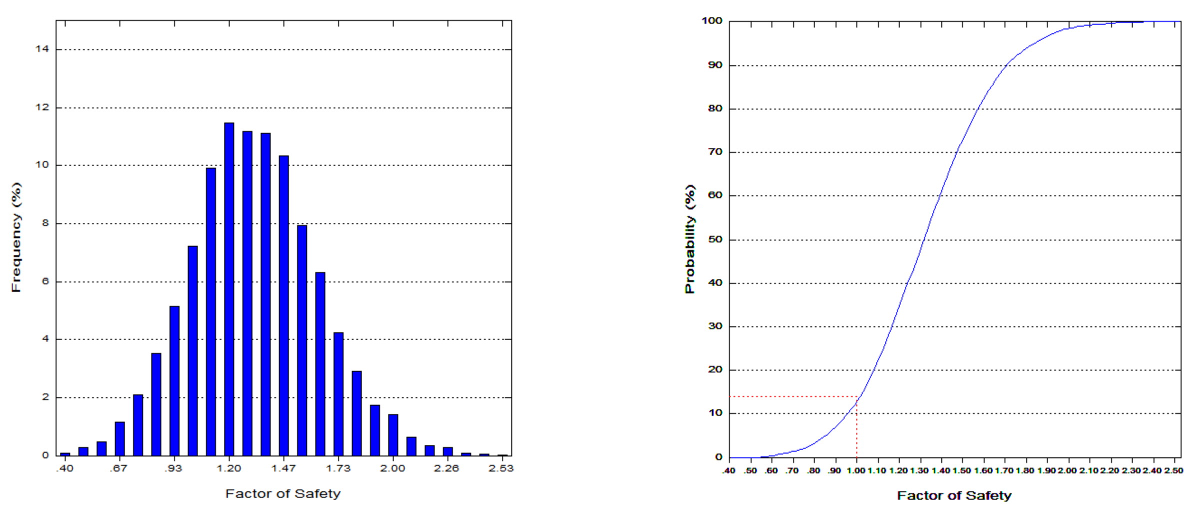


Fig 4.22: MCS with 9999 iterations

Monte Carlos simulation was carried out with all the available methods and failure surfaces in the GALENA software and the results are tabulated in the Tables (4.3 – 4.6).

4.3 Calculation of Reliability Index (β):

The Reliability Index (β) is calculated by the use of the following formula:

$$ß = \frac{\boldsymbol{E(F) - 1}}{\boldsymbol{\sigma(F)}}$$

Table 4.4: Results of Bishop Simplified Method (Defined failure Surface)

Limiting Equilibrium Method	No of Random samples generated	Probability of failure (%)	Mean of Safety Factor	Standard deviation	Reliability Index
Bishop simplified Method (Defined failure Surface)	100	13	1.32	0.263	1.21673
	500	12	1.34	0.292	1.164384
	1000	13	1.33	0.291	1.134021
	2000	13	1.32	0.303	1.056106
	3000	14	1.33	0.304	1.085526
	4000	14	1.34	0.299	1.137124
	5000	13	1.33	0.292	1.130137
	6000	16	1.32	0.299	1.070234
	7000	15	1.33	0.297	1.111111
	8000	15	1.33	0.299	1.103679
	9000	15	1.33	0.301	1.096346
	9999	15	1.33	0.297	1.111111

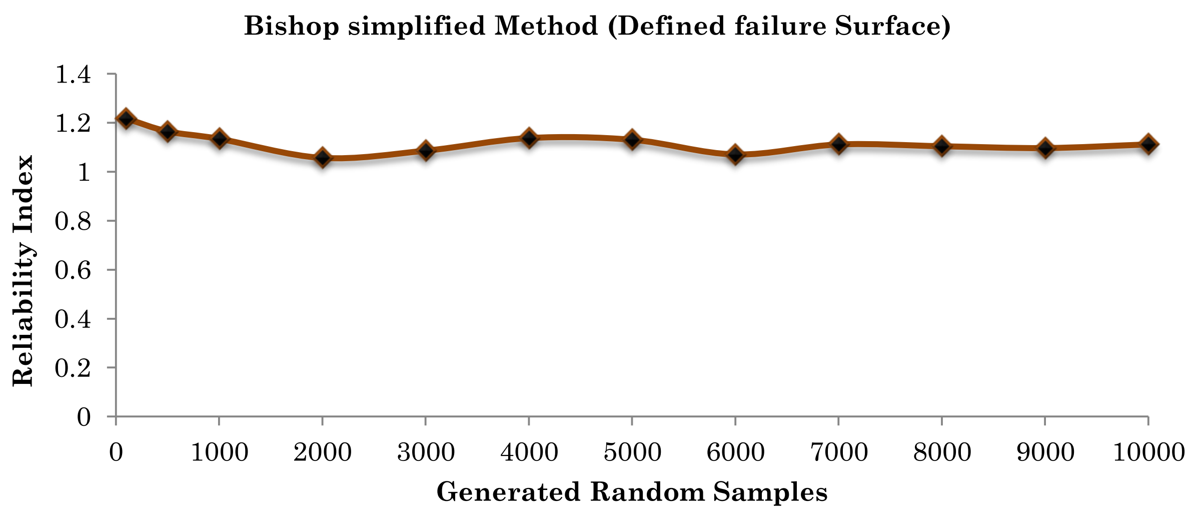


Fig 4.23: Reliability Index vs. Generated Random Samples curve forBishop Simplified Method (Defined failure Surface)

Table 4.5: Results of Spencer Method (Defined failure Surface)

Limiting Equilibrium Method	No of Random samples generated	Probability of failure (%)	Mean of Safety Factor	Standard deviation	Reliability Index
Spencer Method (Defined failure Surface)	100	8	1.36	0.272	1.323529
	500	11	1.35	0.298	1.174497
	1000	15	1.33	0.304	1.085526
	2000	15	1.32	0.297	1.077441
	3000	15	1.32	0.295	1.084746
	4000	14	1.34	0.304	1.118421
	5000	14	1.32	0.299	1.070234
	6000	14	1.33	0.295	1.118644
	7000	14	1.33	0.303	1.089109
	8000	16	1.32	0.302	1.059603
	9000	14	1.33	0.302	1.092715
	9999	13	1.34	0.298	1.14094

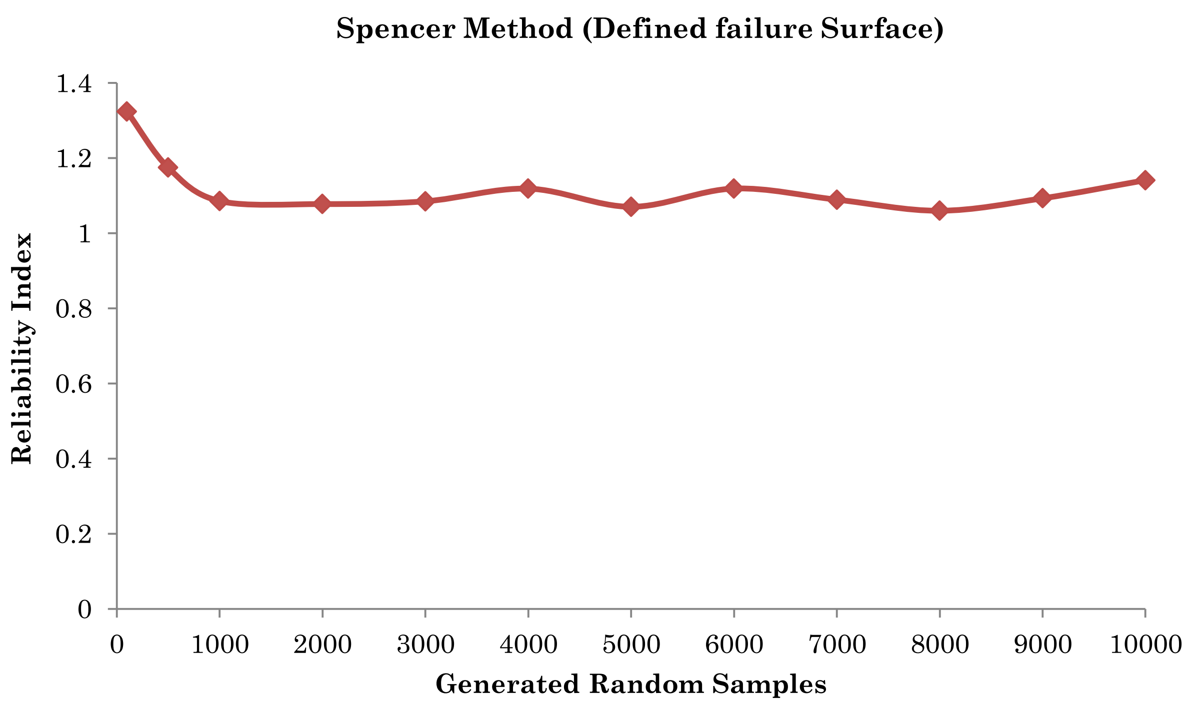


Fig 4.24: Reliability Index vs. Generated Random Samples curve for Spencer Method (Define failure Surface)

Table 4.6: Results of Bishop Simplified Method (Critical failure Surface).

Limiting Equilibrium Methods	No of Random samples generated	Probability of failure (%)	Mean of Safety Factor	Standard deviation	Reliability Index
Bishop Simplified Method (Critical failure Surface)	100	13	1.36	0.305	1.18032787
	500	18	1.29	0.308	0.94155844
	1000	19	1.28	0.299	0.93645485
	2000	18	1.29	0.304	0.95394737
	3000	16	1.31	0.3	1.03333333
	4000	17	1.3	0.299	1.00334448
	5000	18	1.3	0.301	0.99667774
	6000	18	1.29	0.299	0.96989967
	7000	19	1.3	0.3	1
	8000	18	1.31	0.301	1.02990033
	9000	18	1.31	0.3	1.03333333
	9999	19	1.31	0.301	1.02990033

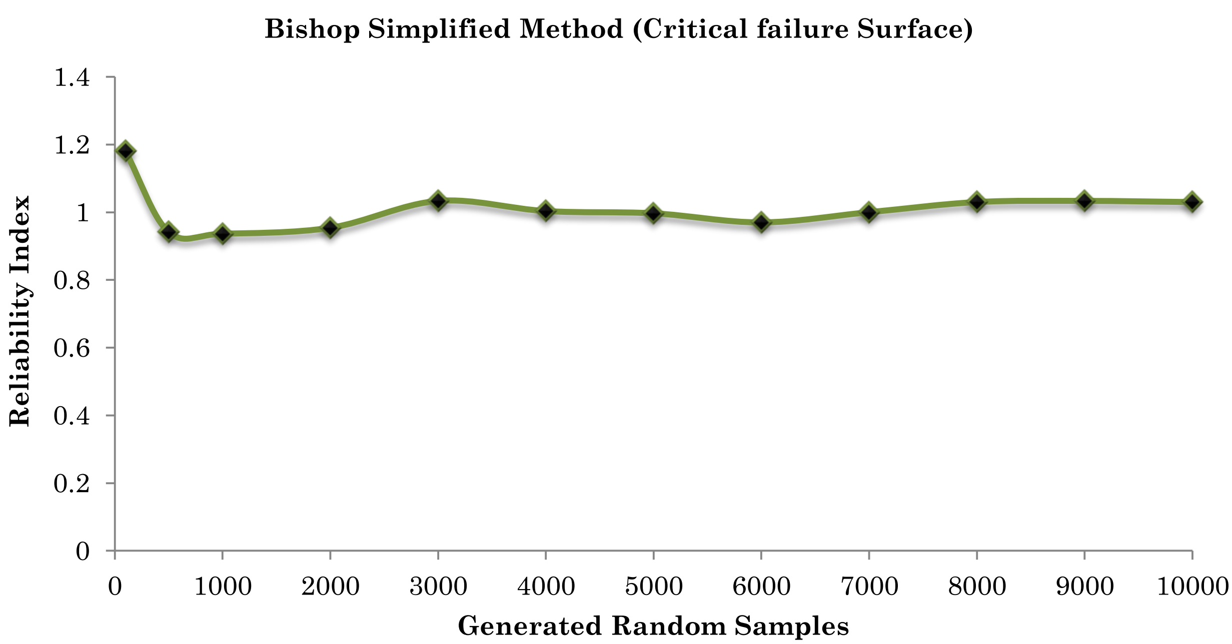


Fig 4.25: Reliability Index vs. Generated Random Samples curve for Bishop Simplified Method (Critical failure Surface).

Table 4.7: Results of Spencer Method (Critical failure Surface)

Limiting Equilibrium Methods	No of Random samples generated	Probability of failure (%)	Mean of Safety Factor	Standard deviation	Reliability Index
Spencer Method (Critical failure Surface)	100	13	1.3	0.293	1.023891
	500	15	1.30	0.294	1.020408
	1000	16	1.31	0.312	0.99359
	2000	18	1.29	0.307	0.944625
	3000	17	1.31	0.309	1.003236
	4000	19	1.3	0.305	0.983607
	5000	18	1.3	0.306	0.980392
	6000	18	1.3	0.305	0.983607
	7000	19	1.31	0.304	1.019737
	8000	18	1.3	0.305	0.983607
	9000	18	1.3	0.305	0.983607
	9999	18	1.3	0.304	0.986842

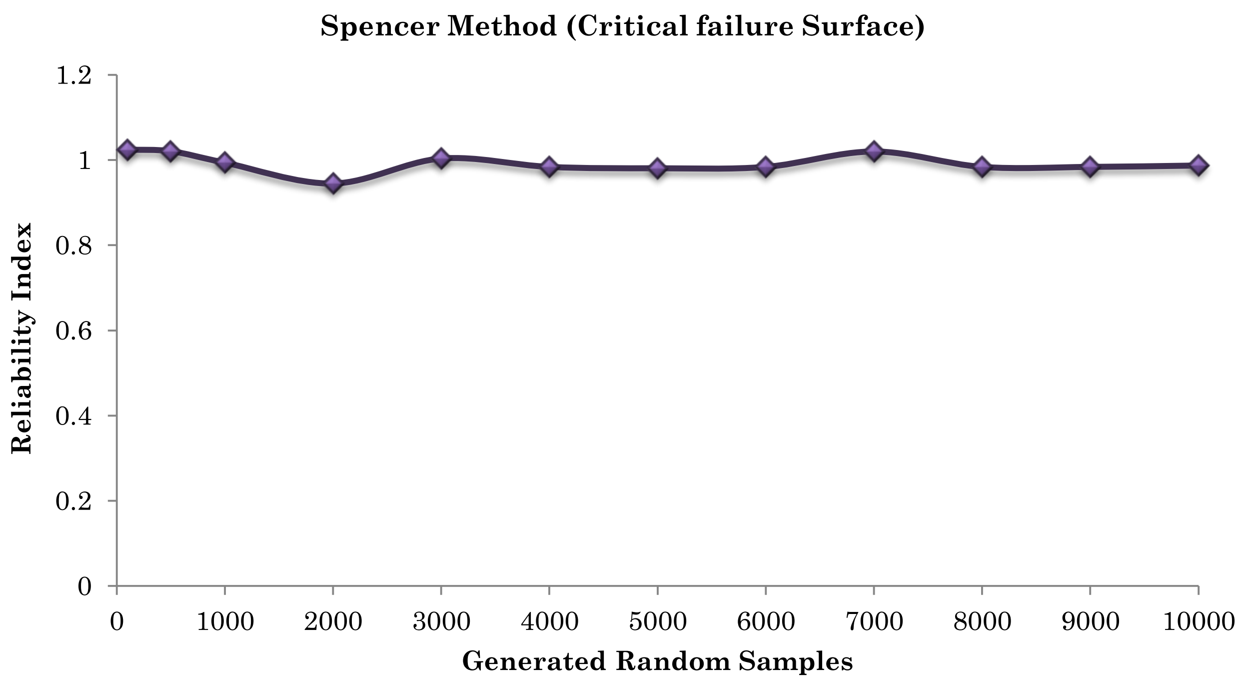


Fig 4.26: Reliability Index vs. Generated Random Samples curve for Spencer method (Critical failure Surface)

It is concluded from the Monte Carlo simulation that, with increase in number of iterations the factor of safety vs. frequency curve (Fig 4.11 – 4.22) takes the shape of normal distribution curve and the reliability index becomes more or less the same. So, it shows the convergence of individual parameters to produce the factor of safety.

Again from the tables (4.3 – 4.6), the variation in values can be found to be:

Table 4.8: The maximum and minimum values of β and P_f.

	Maximum Value	Minimum Value
Reliability Index, β	1.323529	0.936455
Probability of failure (%), P_f	19	8

As observed in Table 4.7andTable 2.4,it can be concluded that the **expected performance level** of the slope under study is **Hazardous.**

4.4 Proposing the optimum slope Height:

The analyses of the existing slopes exhibit one unstable section. So an alternate design has been proposed with the following dimensions and the corresponding safety factor analyses are reported in table. The average data for cohesion and friction has been considered for the analysis.

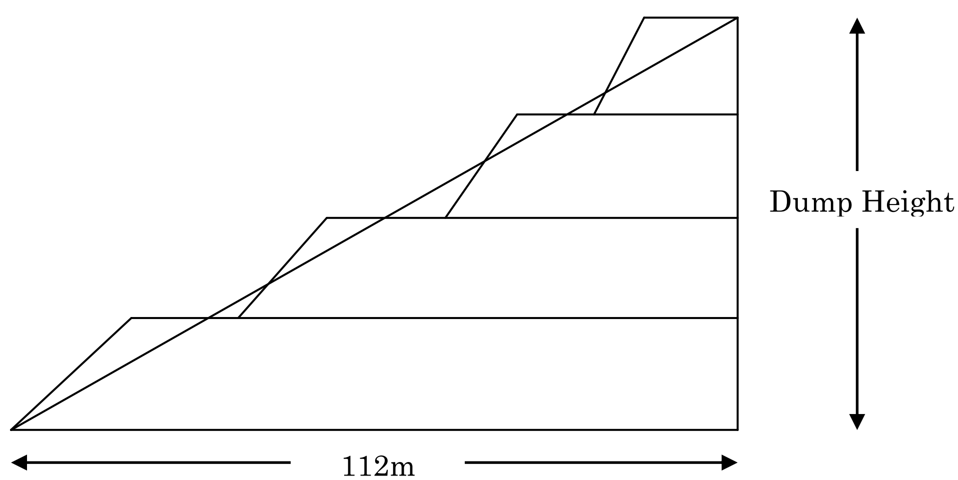


Fig 4.27: Proposed dump site

Hence, considering the dumping of materials commences from the mid-point of the width of the dump, the benches have to be developed as shown in the fig 4.30. Table 4.8 shows some of the analyseswith the Factor of safety for various dump heights.

Table 4.9: Result for Optimum slope Height

Height of the optimum slope	Factor of Safety
90m	0.66
80m	0.67
70m	0.72
60m	0.83
50m	0.98
40m	1.22

Hence, the optimum slope height of the dump = 40m. To carry out safe operating condition inside a mine, the height of the slope should be limited to 40m.

4.5 Proposed Individual Bench Design:

The angle of inclination of the bench = 24^0 (It is close to the frictional angle and less than the angle of repose).

Tension crack is assumed to be situated at 3m from top of the surface to simulate the worst case scenario.

The benches are designed in such a manner that, the maximum width in one side is restricted to 112m.

Table 4.10: Stability analysis of the proposed bench.

Layout No	Bench Height, m	No of benches possible	Overall Factor of Safety, F	Comments on Stability
1	8	4	1.56	Stable
2	10	3	1.46	Stable
3	12	3	1.8	Stable
4	14	2	1.38	Stable
5	16	2	1.29	Stable

CHAPTER 5

CONCLUSION AND RECOMMENDATION

5.1 Conclusion:

The following conclusions have been drawn from the investigation.

- The dump contains heterogeneous materials, whose properties vary throughout the dump.
- At places large sized boulders (> 60cm) are dumped in these sections. While dumping, air gap remains between them. During rainy season, the pit top water moves through these areas. Hence, the slope becomes potentially weak.
- The factor of safety for the sections AA', BB' and DD' are greater than 1.3, which shows the stability of those regions. But at section CC', the factor of safety is found to be 0.86, confirming that the section has already failed.
- The tension crack was assumed to be situated at 3m from the slope surface. Since, multiple restraint analysis is done; factor of safety is not affected much by the position of the tension crack in the slope.
- Though Monte Carlo Simulation concludes that the slope is in a Hazardous condition, but field visits inferred no such results. This can be explained by the following reasons:
    1) The distribution of cohesion of bench 'A' generate some negative values, which are not included in the simulation, thereby increasing the no of samples which fail to produce a result.
    2) The Direct Monte Carlo simulation is successful if the sample sizes are greater than 10,000, which is not possible in Galena.
    3) The Bessel's correction was not applied despite the sample sizes for practical purposes being less than 30.
    4) Laboratory analysis of lesser number of samples.

5.2 Recommendation:

In this investigation, a few prospects of slope stability; as slope of the bench, height of bench as well as angle of internal friction, cohesion and density of the material has been determined. However there are many factors that affect the slope stability such as ground water table, grain size of the dumped material, etc. So it is strongly recommended that the following may be taken into consideration in future.

- The samples were not taken to the complete depth; the samples have been taken just from 2-3 feet depth and they do not represent the exact field conditions.
- Proper provisions have to be made for the channelization of the rain water by constructing the suitable drains surrounding the dump yard.
- Segregation of the dump has to be done properly i.e. according to the size of the disposed material. The fines should be dumped separately, and so the boulders.
- In the analysis, the phreatic surface should have been included.
- Though Limit equilibrium method is used for analysis, sometimes it fails to produce the accurate result. So, Finite equilibrium method may be considered for the analysis and evaluate its effectiveness in the analysis.

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